WASHITA RIVER WATERSHED RESEARCH FINDINGS

April 1968

U.S. Department of Agriculture
Agricultural Research Service
Soil & Water Conservation Research Division
SOUTHERN PLAINS WATERSHED RESEARCH CENTER
Chickasha, Oklahoma 73018

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RESEARCH FINDINGS

April 1968

Reported by Monroe A. Hartman at the 63rd meeting of the Arkansas-White-Red Basins Interagency Committee, Austin, Texas

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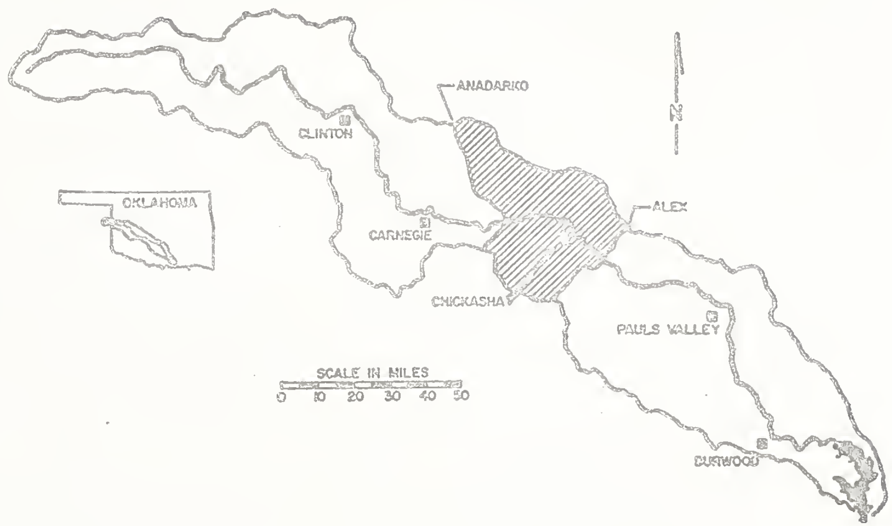
The research discussed in this publication is being conducted cooperatively by the Agricultural Research Service, Oklahoma State University, University of Oklahoma Research Institute, Oklahoma Water Resources Board, Environmental Science Services Administration, and other Federal, State, and local agencies.



Location of the Southern Plains Watershed
Research Center on the Washita River Basin

WASHITA RIVER WATERSHED RESEARCH FINDINGS

INTRODUCTION

The work of the Southern Plains Watershed Research Center is continuing to move ahead dynamically. Typical of our time, 1967 has been a year of change--a year that has produced new problems and fresh research challenges, like the water quality problems.

This report highlights our research activities and accomplishments. In 1961 the Agricultural Research Service started a hydrologic research study of a 1,130-square-mile area in a central segment of the Washita River Basin in Oklahoma to determine the change in downstream runoff, sediment flows, and groundwater levels when flood control measures are applied to the uplands. These measures include land treatment, but the dominant feature is the flood-detention reservoir. Detailed rainfall, runoff, sediment yield, and groundwater data are being collected and analyzed. These findings are of great interest to water-short areas such as Oklahoma.

The results reported here are based on detailed field observations being made by the Agricultural Research Service in cooperation with State and Federal agencies at selected locations in a portion of the Washita River Basin in west-central Oklahoma. The attached figure shows the portion of the Washita River Basin selected for study.

RAINFALL PATTERNS IN THE RESEARCH AREA

Arlin D. Nicks

A dense rain gage network has been installed on the 1,130-square-mile study reach. The basic network, containing 168 recording rain gages spaced on a 3- by 3-mile square grid pattern, provides needed data for the major tributary watersheds and the entire study reach. The grid pattern for the basic network is shown in Figure 1. Additional gages have been installed in a still denser pattern on smaller watersheds, which range in size from a few acres to 7.6 square miles.

The rain gage network is used to obtain an accurate estimate of area rainfall and a definition of storm patterns through point measurements. Occasionally, the runoff response from a storm has indicated that a rainfall pattern may have occurred which was different from that defined by the network. Additional point measurements between stations of the existing network could have defined the correct rainfall patterns.

Figure 1
The rain gage distribution over the study reach

The normal annual rainfall for the region according to Weather Bureau records (30-year period, 1931-1960) is 31 inches per year. However, for the past six years the rainfall has been below normal. Table 1 shows the annual rainfall for the network for the period 1962 to 1967:

Table 1.--Annual Rainfall

Year	Amount (inches)
1962	28.44
1963	19.91
1964	28.77
1965	26.40
1966	21.69
1967	26.41

During this entire 72-month period only 18 months have had rainfall above normal.

Weather Bureau records indicate that annual rainfall varies from from 35 to 28 inches in an east to west direction across the study reach. The observations during the past six years show that there was an extreme variation of 6.32 inches across the region; however, the minimum rainfall was in the southwest and the maximum in the northeast. Figure 2 shows the average annual rainfall over the region determined from the measurements obtained from the rain gage network.

The major portion of the annual precipitation is received during the period of March to October, with peaks occurring in May and September. Both convective and frontal storms have produced a wide variation in rainfall between points closely spaced on the network. Differences as large as 4.5 inches in 6 miles have been observed in both local and widespread storms[1]. Yearly occurrence of such variations appears to be characteristic of rainfall for this climatic region.

Because of the large variation in rainfall over such a short distance, a study was made to determine the adequacy of a network for assessing the mean daily rainfall over the entire study area. The results indicated that at the 5-percent level, the average rainfall for the area computed from 5 gages was significantly different from the average from 158 gages. At a density of 10 equally spaced gages, there was no significant difference, indicating that daily average rainfall estimates for the 1,130-square-mile area could be obtained from a network that is less dense than that installed.

[1]Nicks, A. D. and Hartman, M. A. Variations of rainfall over a large gaged area. Transactions ASAE, Vol. 9, No. 3, pp 437, 439. 1966.

Figure 2
Average annual rainfall in inches
1962-1967

The 10-gage network might supply an adequate estimate of the daily rainfall for the 1,130-square-mile drainage area, but for small areas in the basin a much denser network is required. A good example of this need is the storm that occurred May 20, 1966 in the southwest corner of the network between the cities of Anadarko and Chickasha. The network observations indicated the rainfall pattern shown in solid lines on Figure 3. By including a 3.2-inch rainfall amount reported by a resident in the area, the rainfall pattern would be like that shown by the dotted lines in Figure 3.

Streamflow rises sometimes indicate heavy rainfalls occurred in the watershed that were not recorded on any of the gages within the network. An example of this is the storm of September 3, 1966 in the upper West Bitter Creek watershed. A 5-foot rise occurred at the gaging station on the night of September 6, yet the rainfall data from the network did not indicate that such a rise was possible. So an effort was made to collect additional rainfall data within the area. The measurements from a number of 5-inch glass rain gages located within the network were added to those of the network gages. The map in Figure 4 shows the resulting rainfall pattern for both the network and the glass gage data.

Shown in Figure 5 is the storm isohyetal pattern of a record rainfall event that occurred on Sugar Creek watershed in the northwest corner of the network. The pattern shown was produced by a line of thunderstorms along a solid front that moved across Oklahoma. This storm, which followed two days of near uniform rainfall with accumulations up to 4 inches, caused flash-flooding and considerable property damage[2].

A noteworthy characteristic of this storm was the high rainfall intensity recorded at various stations. Table 2 shows the maximum amount recorded for time intervals of 5 minutes to 6 hours. In most instances, these amounts exceeded the maximum recorded for the state of Oklahoma[3]. The clock-hour distributions are shown in the maps on Figure 6.

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[2]Climatological Data National Summary. Weather Bureau, Vol. 16, No. 9 September 1965.

[3]Maximum Station Precipitation for 1, 2, 3, 6, 12, and 24 hours. Part XXVI Oklahoma, Weather Bureau Technical Paper No. 15. 1961.

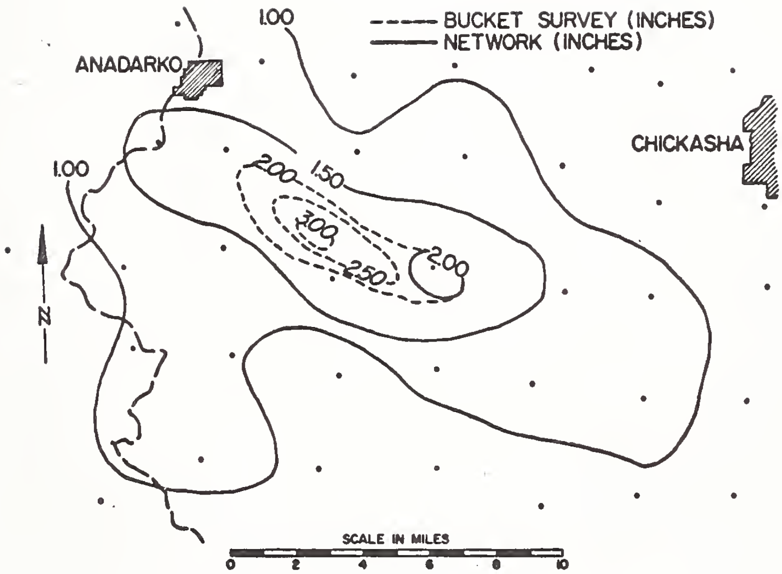

Figure 3
Isohyetal map of May 20, 1966 storm

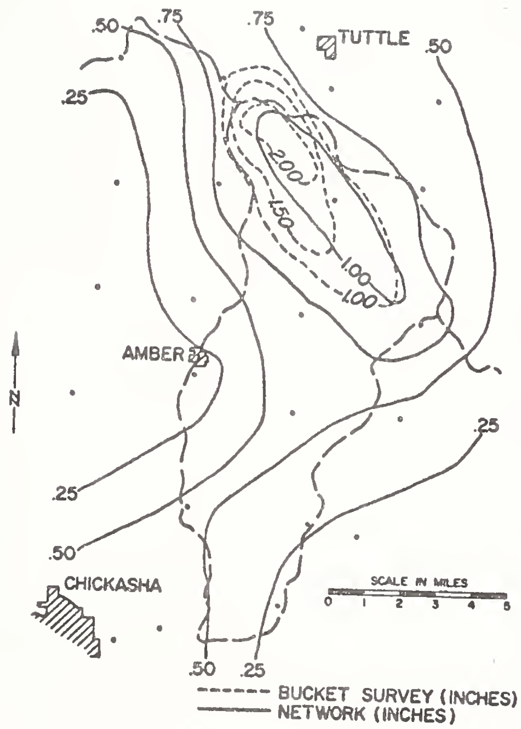

Figure 4
Isohyetal map of September 3, 1966 storm

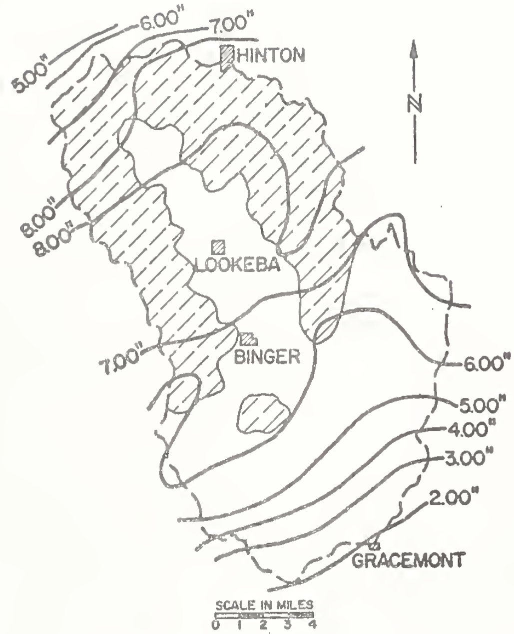

Figure 5
Storm rainfall map for Sugar Creek watershed
September 20, 1965

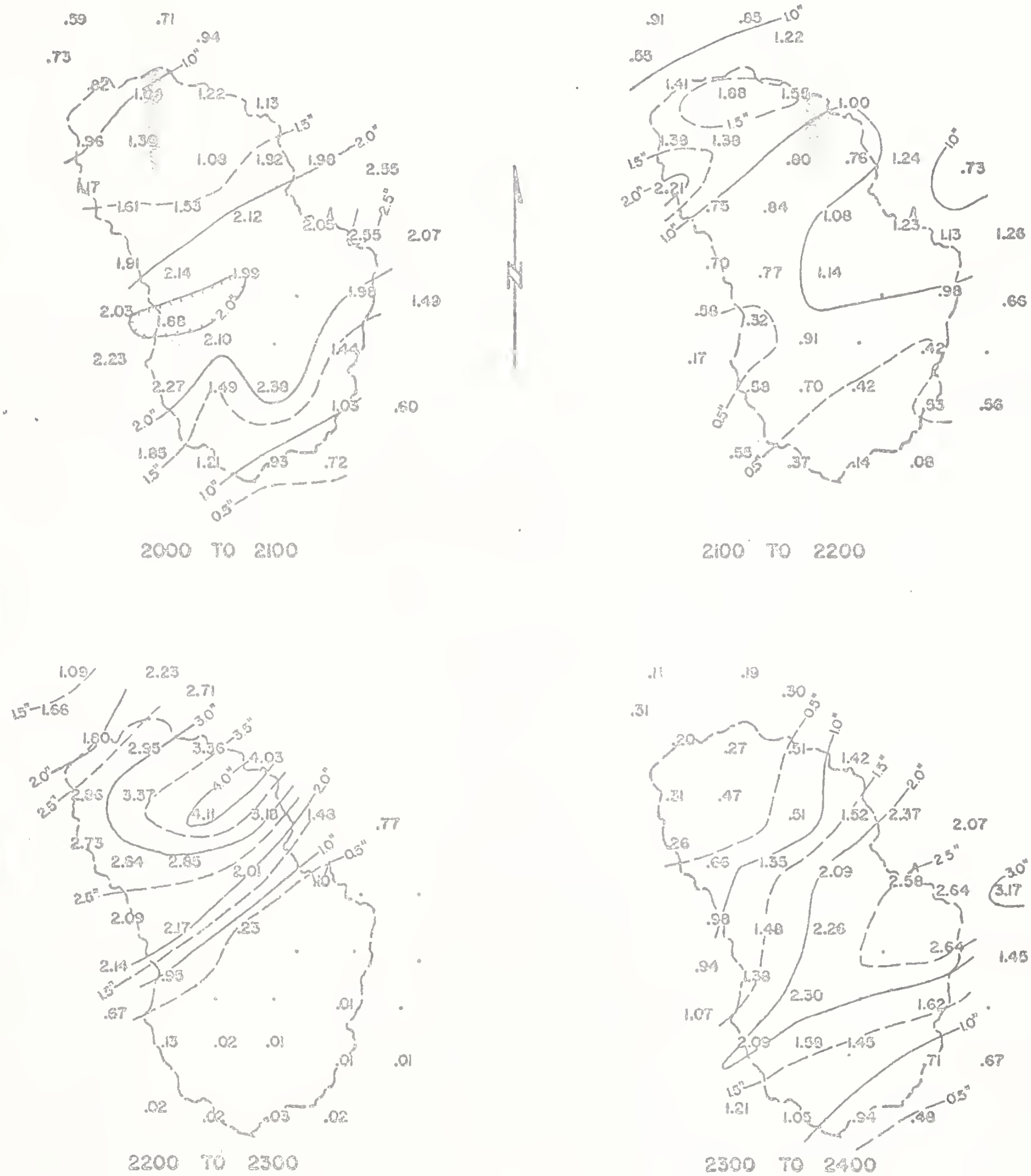

Figure 6
Hourly rainfall for September 20, 1965 storm on Sugar Creek watershed

Table 2.--Maximum intensities for selected time intervals - storm of September 20, 1965

Time interval	Amount (inches)	Intensity (inches/hour)
5 min.	.83	9.96
10 min.	1.41	8.46
15 min.	1.68	6.72
30 min.	2.64	5.28
1 hr.	4.61	4.61
2 hr.	5.44	2.72
6 hr.	8.83	1.47

Although the rainfall pattern appears to vary uniformly north-to-south across the watershed, the accumulations resulted from intense cells moving at high speeds. Time-lapse radar-scope film of the storm obtained from the National Severe Storm Laboratory at Norman, Oklahoma[4] showed the movement of small cells along the storm line to the northeast. Figure 7, a photograph of the PPI scope from the Norman radar, shows the location of the storm with respect to the boundary of the network at 2000 hours. Tracking the storm movement by tracing consecutive frames of the radar film reveals an average cell speed of 30 miles per hour to the northeast. A similar tracking was made of the surface rainfall by plotting average 10-minute intensities computed from the network gages. Maps of these intensities for a 90-minute period in the early stage of the storm are shown on Figure 8. Two cells were traced by these methods, the first occurring from 1850 to 1920 hours near the top of the watershed, and the second cell traced was from 1950 to 2020 hours in the southern part of the watershed. The tracks shown in Figure 9 were established by plotting the center of these cells and measuring the distance and direction of movement. Tracks derived from the radar photographs and from the rain gage network agree quite well in both speed and direction of movement. In both instances the average speed of the cells was 30 miles per hour to the northeast.

Figure 10 shows a plotting of rainfall intensity versus time for gages along the line of the cell tracked from 1950 to 2020 in Figure 9. Gage 51 is located at the southwest end of the reach and gage 16 is at the northeast end. The peaks in this illustration are assumed to represent passage of cell centers over the gages. Though it is likely that the centers of the cells did not pass directly over the gages, it is believed that the observed intensities are satisfactory for tracing cell movement.

[4]Kessler, Edwin. Radar measurement for the assessment of areal rainfall: Review and Outlook. Water Resources Research, Vol. 2, No. 3. 1966.

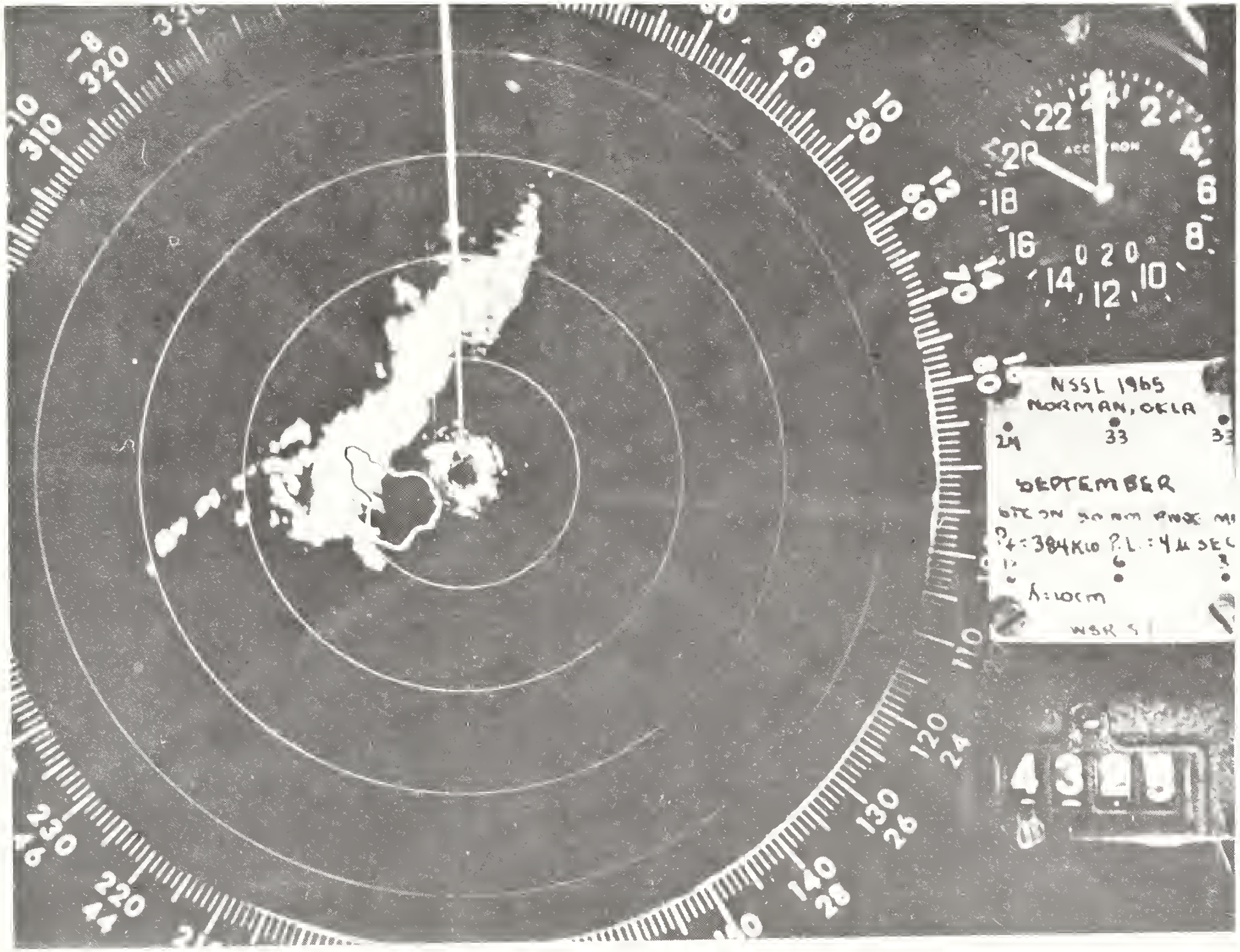

Figure 7
PPI scope photograph of September 20, 1965 storm at 2000 hours
(From National Severe Storms WSR-S7 radar)

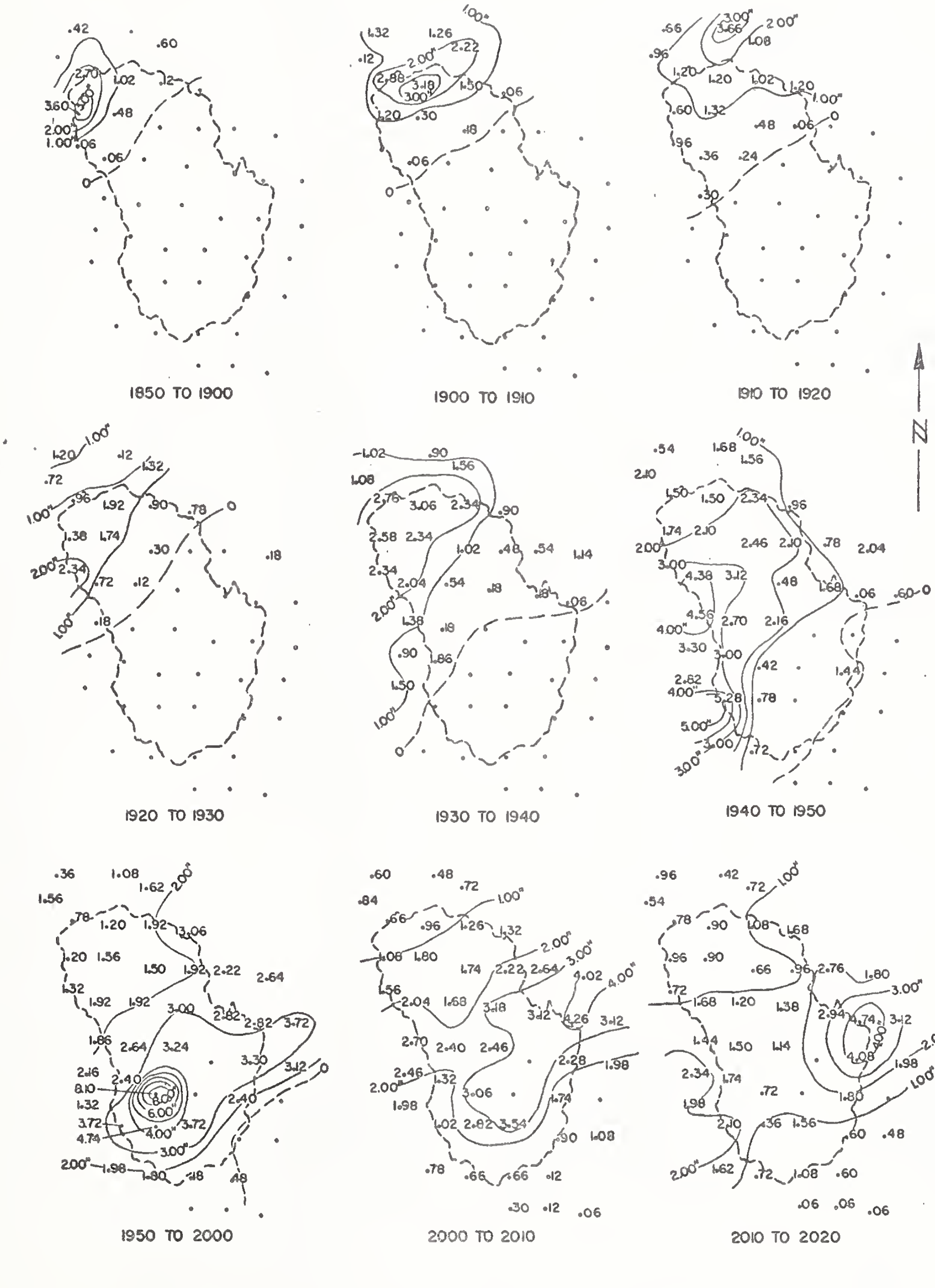

Figure 8
10-minute intensity (inches per hour) plotting of storm rainfall
September 20, 1965

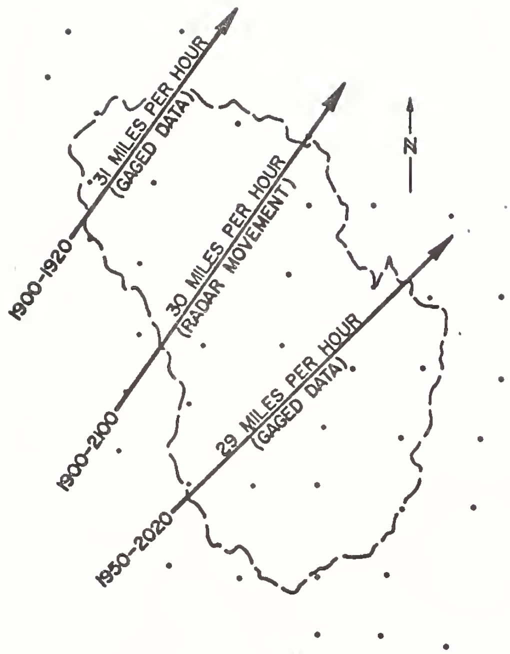

Figure 9
Cell tracks from radar and rain gage network data

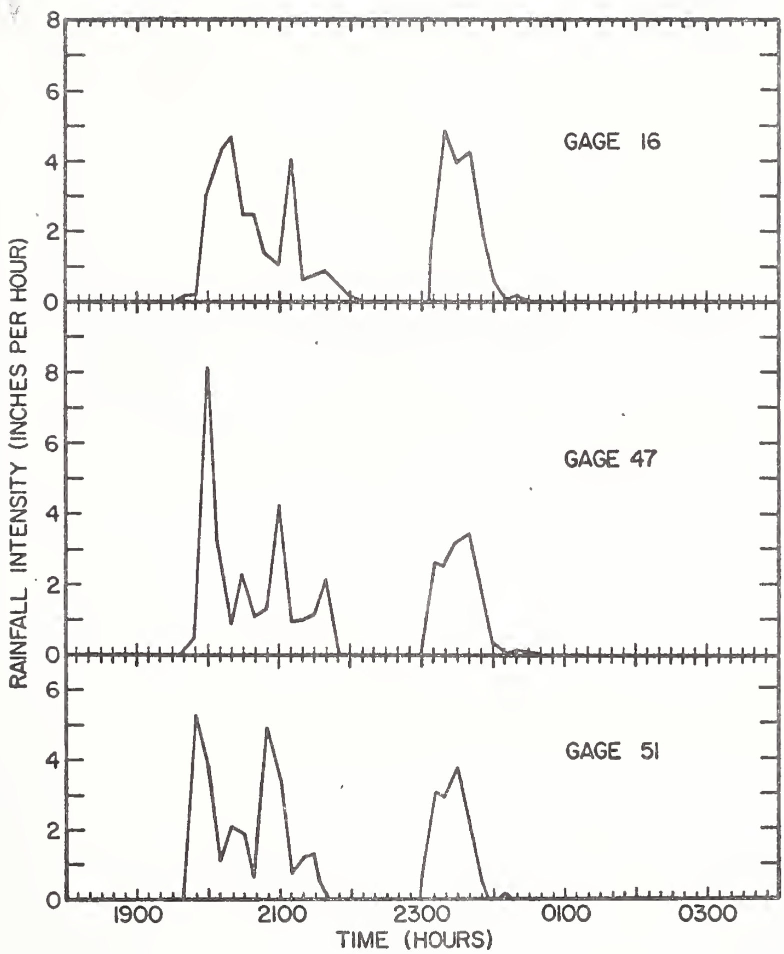

Figure 10
Rainfall intensity plots for selected gages along the track of cells of the storm of September 20, 1965

Summary and Conclusions

Some small isolated storms occurring in the Southern Plains climatic zone are small enough to pass between stations of a 3- by 3-mile gage spacing. These undetected storms can cause inconsistencies in the rainfall-runoff relationship for a watershed. The frequencies of occurrence of these events and their hydrologic significance are not known at present. Additional data are needed so that better analytical techniques and procedures for projecting point rainfall to areal estimates can be developed.

WATERSHED RUNOFF

Russell R. Schoof

Instrumentation and Data Acquisition

Since the beginning of this research project in 1961, mean daily flows for about 100 station years of record from 20 main-stem and tributary gaging stations have been compiled. The present gaging network shown in Figure 11 includes 5 stations on the Washita River and 14 stations on tributaries.

Performance of the bubble gage water level recorders has been improved by installing the orifice in a giant sand point[5], 4 inches in diameter and 5 to 20 feet long. The large sand point provides ample storage for sediment that enters during storm flows and adequate anchorage for the orifice. A self-cleaning sand point can be constructed from an open-bottomed, short length of pipe fastened to a bridge pier at an elevation such that the stream bed scours below the bottom of the pipe during a rise.

Discharge is measured with the Price current meter on a rod suspension when the flow can be waded, and on a cable suspension when the flow can not be waded. Field tests indicate that approximately 4.3 percent less discharge is measured with rod suspension than with cable suspension when the same meter rating table is used[6]. Previous investigators using data from laboratory calibrations of current meters found a difference of 2 percent in the opposite direction.

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[5]Hunt, Charles G.; Price, Harold E.; Schoof, Russell R.; and Hartman, Monroe A. A giant sand point for bubble gage orifice housing. USDA, ARS 41-129, 1966.

[6]Schoof, Russell R. Stream gaging precision related to meter suspension and procedural error. Multilithed. Master's thesis, Oklahoma State University, May 1965.

Figure 11
Gaging station locations on the Washita River study reach

The reasons for the unexpected result are not known. However, when comparing the result with the findings of other investigators note that in this experiment the tests were conducted under field conditions. In the field the stream gager stands in the water when making a rod suspension measurement, whereas in the laboratory rating the meter is suspended from a tow car and pulled through still water. Even though, in the field, the stream gager stands in a streamlined position at arm's length from the wading rod, his body may still retard the flow in the vicinity of the meter.

A system for computation, checking, and routine analysis of stream flow and sediment transport data has been developed[7]. Economy is maximized and human error is minimized by using electronic data processing equipment including a chart reader, card keypunch, computers, and card sorter. The system flow chart, shown in Figure 12, includes 12 Fortran computer programs and is applicable to weir control and channel control gaging stations with or without loop ratings.

Precipitation and runoff have been below normal on the 1,130-square-mile study area since the initiation of the research project in 1961. Runoff from the study area has been less than 1 inch each year since 1962. The normal, long-term, annual runoff for the Chickasha area is 3 to 4 inches, and the 30-year normal precipitation is 31 inches at Chickasha. Average precipitation on the study area the past six years has been 25.6 inches per year. Annual precipitation and runoff for the period of record of each watershed is shown in Table 3.

Flow Losses and Abstractions

To supply a request from farmers for irrigation water, about 1,950 acre-feet of water were released by the Fort Cobb Conservancy District from Fort Cobb Reservoir, about 44 channel-miles upstream from Anadarko, during August 1964. This release into the Washita River, which was nearly dry, provided an opportunity to obtain data on channel losses within the 74 river-mile study reach from Anadarko to Alex[8].

The water was released at 150 cubic feet per second for the first two days. The leading edge of the flow moved through the study reach at an average velocity of 0.93 mile per hour. The average loss rates

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[7]Schoof, Russell R.; Seely, Edward H.; Nicks, Arlin D.; Edens, Carlton D.; and Allen, Paul B. Electronic data processing system for stream flow. Pending publication.

[8]Schoof, Russell R. and DeCoursey, Donn G. Conveyance of irrigation water in a natural channel. Published in the Proceedings of the Second Annual American Water Resources Conference, November 20-22, 1966.

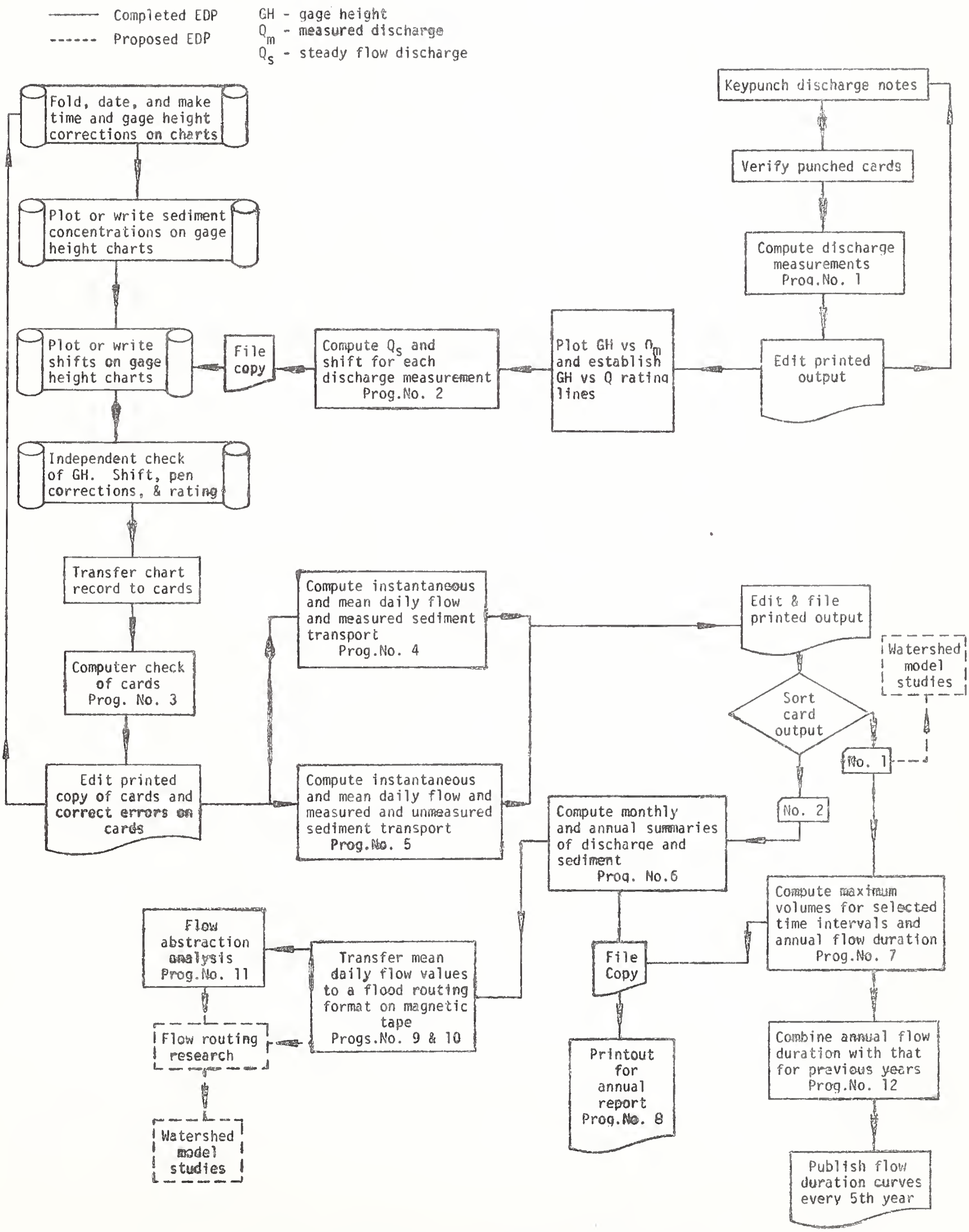

Figure 12 Data Processing Flow Chart
for Streamflow and Sediment Transport

Table 3.--Annual precipitation and discharge of main stem and tributary watersheds

Watershed		1962		1963		1964		1965		1966		1967	
No.	Area (sq.mi.)	P (in.)	Q (in.)	P (in.)	Q (in.)	P (in.)	Q (in.)	P (in.)	Q (in.)	P (in.)	Q (in.)	P (in.)	Q (in.)
100	3,656		1.513		.498		.792		1.626		.508		.306
110	39.3					30.80	.176	25.04	.137	25.04	.000	24.55	.000
111	26.0			17.07	.997	31.60	1.452	20.85	.850	19.73	.543	24.39	.537
121	201.5	26.28	1.060	25.22	.525	25.50	.354	32.02	2.153	21.33	.624	24.87	.592
131	40.1			18.99	.705	31.90	.552	24.22	.577	20.18	.243	26.53	.303
200	4,083		1.461		.507		.753		1.554		.541		.324
311	23.8											27.84	2.317
400	4,259		1.456		.504		.694		1.452		.530		
411	53.4			19.04	.337	29.10	.372	23.93	.437	20.77	.069	25.41	.317
500	4,328						.692		1.393		.531		.354
511	60.8			20.37	1.297	28.90	.943	24.35	1.234	24.00	1.556	27.27	1.549
512	35.6					33.30	2.266	25.29	2.121	24.69	1.856	28.14	1.828
513	19.6							25.16	2.151	25.16	2.085	28.56	2.312
522	207.7	30.45	2.943	17.72	1.051	30.80	1.733	25.69	1.240	19.60	.658	26.33	.685
600	4,707						.750		1.403		.565		.408
611	7.57	26.65	2.332	17.90	.945	31.70	1.896	22.14	.499	24.29	.748	24.81	.411
612	.88	28.39	4.729	18.96	1.555	29.40	.653	24.04	.084	24.74	.313	25.53	.324
621	33.3					33.10	2.883	26.98	2.204	22.14	1.113	27.21	1.212
700	4,787		1.598		.552		.790		1.385		.554		.420
Study Reach	1,130	28.44	2.000	19.91	.730	28.77	.783	26.40	.614	21.69	.704	26.41	.791

within subreaches of the study reach varied from a negative loss (a gain) to a loss as high as 4.39 acre-feet per mile day. The average loss rate from Fort Cobb to Alex was about 0.58 acre-foot per mile day. Only about one-fourth of the water released was diverted from the channel for irrigation. Most of the remainder was lost by seepage or evapotranspiration. A small amount was in channel storage when a storm broke the drought.

Streamflow losses in the Anadarko to Verden reach of the Washita River were also measured during natural storm flows. Sixteen runoff events with peak mean daily inflows ranging from 500 to 10,000 cfs showed flow losses varying from 5 to 77 percent of the daily inflow[9]. However, some of the water lost during the peak returned to the stream during the recession. Figure 13 portrays three of the runoff events and also shows the accumulated flow loss (abstraction) or gain during a six-month period. An upward trend in the accumulated abstraction or gain curve indicates flow returning to the stream from temporary storage in banks and flood plain soils.

A detailed analysis of the surface water budget for the period September 20-30, 1965 for the study reach is shown in Table 4. The average loss rate of 27.9 acre-feet per mile per day was much higher than the loss rate observed during the release of water from Fort Cobb because the flow was much deeper in the stream during the natural event than it had been for the period of water release. High flows, which saturate the banks, reduce abstraction from succeeding flows. This is observed in three runoff events. Also, abstractions from storm runoff events were greater in the summer months when evaporation loss was great and the antecedent flow was low. Maximum gains were associated with the winter and early spring flows.

Hydrologic Influences of the Sugar Creek Flood Control Program

The effects of an upstream flood prevention program on the hydrology of Sugar Creek, a 240-square-mile tributary to the Washita River, were studied[10]. Figure 14 shows hydrographs for three storms occurring after the installation of detention reservoirs and the expansion of improved land use measures compared with hydrographs for the same storms estimated for the pretreated conditions. The storm of

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[9] Schoof, Russell R.; Hartman, Monroe A.; and Hunt, Charles G. Determining streamflow abstractions from antecedent conditions. Presented at the General Assembly of the International Union of Geodesy and Geophysics, Berne, Switzerland, September 25-October 7, 1967, and submitted for publication in IUGG Transactions.

[10] Hartman, Monroe A.; Ree, William O.; Schoof, Russell R.; and Blanchard, Bruce J. Hydrologic influences of a flood control program. Published in the Journal of the Hydraulics Division, Proceedings of the ASCE, Vol. 93, No. HY3, May 1967.

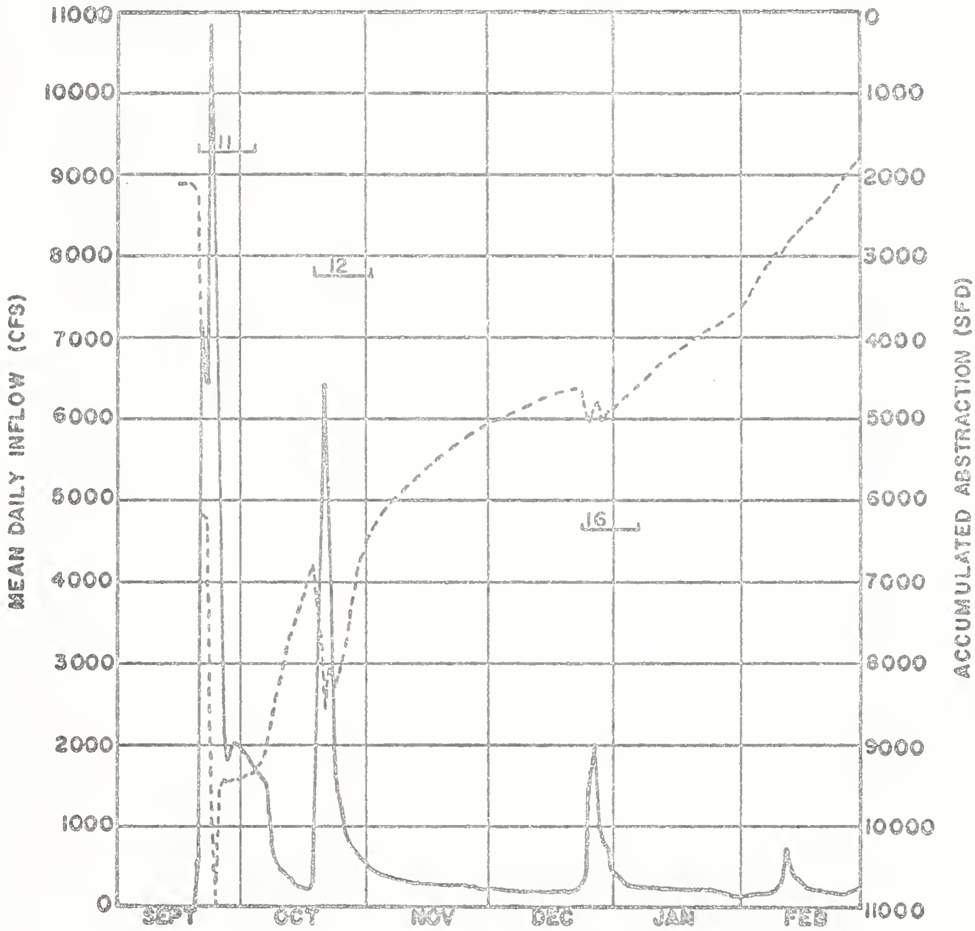

Figure 13
Anadarko-Verden reach inflow and accumulated abstractions and gains for selected events in 1965 and 1966

Table 4.--Washita surface water budget (acre-feet) for the period September 20-30, 1965

Station	Inflow -	Outflow -	Channel storage =	Loss	River miles	Loss acre-feet/mi/day
Anadarko	85,096		1,362		21.8	
Sugar	13,981		53		11.0	
Verden		83,323		14,339		39.7
Verden	83,323		1,936			
Line	54					
Turnpike		71,744		9,697	26.9	32.8
Turnpike	71,744					
W. Bitter	123					
E. Bitter	155					
L. Washita	246					
Winter	240					
Alex		66,892	2,086	3,530	30.0	10.7
Anadarko-Alex	99,895	66,892	5,437	27,566	89.7	27.9

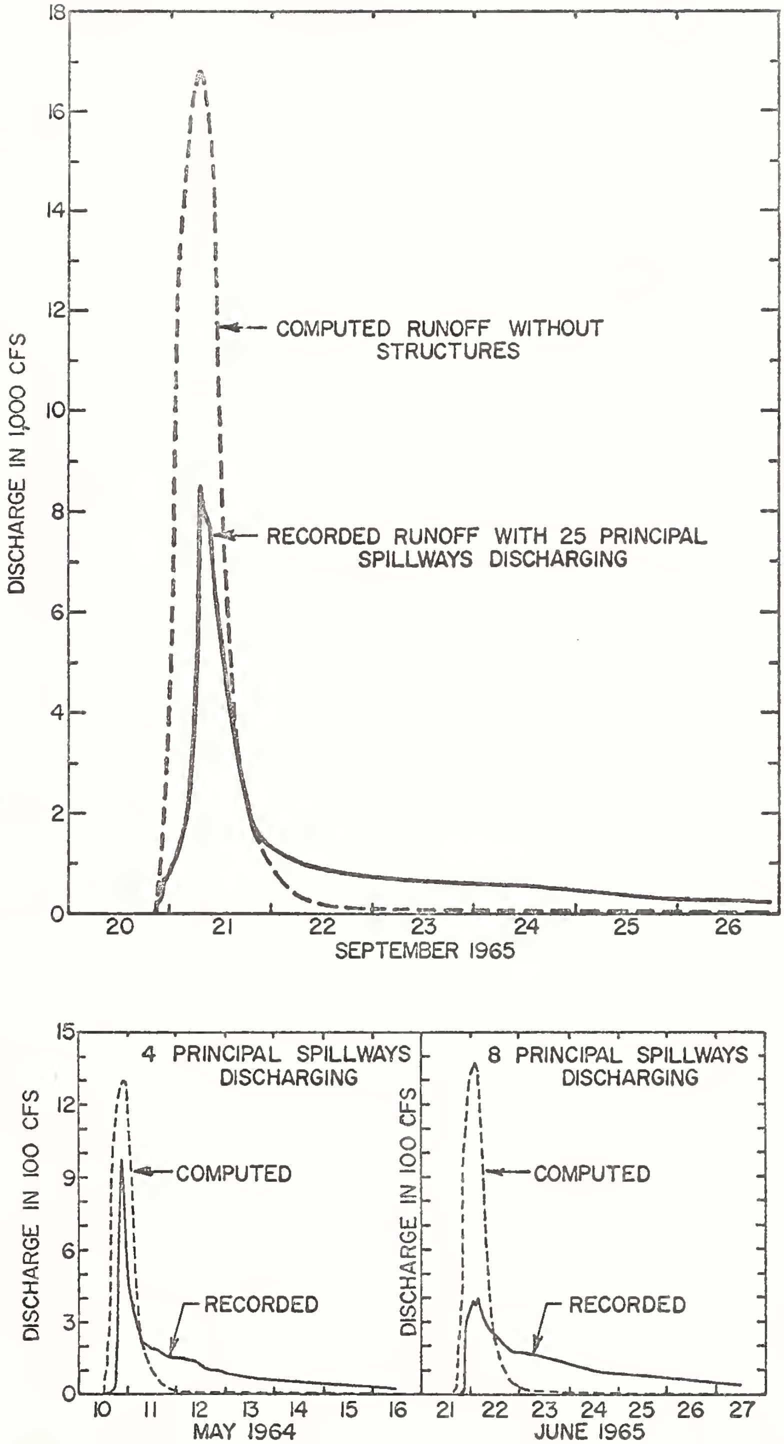

Figure 14
Comparison of Sugar Creek hydrographs

September 1965 nearly equaled the design storm and provided a severe test of the flood control works. Flood peaks were reduced at least one-half by treatment. The recession portions of the after-treatment hydrographs were lengthened and flattened. The character of the recession flow was influenced by the number of detention reservoirs discharging into the stream following each individual storm. The volume of water infiltrated was reduced on the flood plain and increased in the detention reservoirs.

UNIT SOURCE WATERSHED RUNOFF STUDIES

Edd D. Rhoades

Despite the limitations inherent in relating the performance of the isolated small watershed to its performance as a unit in a large complex watershed, the small experimental watershed is still a useful tool for the research hydrologist. He selects them carefully so that they are representative of a particular condition. By manipulating the cover on a small watershed, he can study the effect of land management on its hydrologic performance; or he can use the experimental watershed to evaluate the runoff-producing characteristics of a particular geology-soil-cover complex.

The Southern Plains Watershed Research Center has selected eight small grassland watersheds and eight small cropland watersheds for intensive investigation of the effect of land use on hydrologic performance. Detailed soil surveys and topographic surveys were made to define their physical features. Rainfall is measured with one or more recording rain gages on each small watershed; runoff is measured with V-notch weirs or flumes, or by observing storage changes in farm ponds; and soil moisture in the surface 4.0-foot profile is measured with a neutron soil moisture probe.

Four of the grassland watersheds (R-1, R-2, R-3, and R-4) range in size from 18 to 25 acres and are located about 17 miles northwest of Chickasha, Oklahoma, on a permeable, well-drained, fine sandy loam. They are adjacent to each other and are all within the same general pasture area. Portions of these watersheds were cropped several years ago. Although there has been considerable erosion, there are no gullies or active erosion on these watersheds now. Runoff is collected in a farm pond at the lower side of each watershed. The volume of runoff is calculated from the stage-volume relationship of each pond.

The other four grassland watersheds (R-5, R-6, R-7, and R-8) range in size from 18 to 27 acres and are located about 15 miles northeast of Chickasha, Oklahoma, on soils that are loamy to silty clay loam with good surface drainage, but poor internal drainage. Runoff is measured through V-notch weirs. Watersheds R-5 and R-6 are native grassland sites; watersheds R-7 and R-8 were formerly cultivated, but were changed to grassland after they had eroded badly.

The eight cropland watersheds are located in the Washita River alluvium and range in size from 13 to 44 acres. Watersheds C-1, C-2, and C-7 are dryland row cropped, primarily in cotton. Watersheds C-3 and C-4 are in irrigated cotton; C-5 and C-6 are in wheat; and C-8 in two-year-old alfalfa.

Discussion

Grassland - Table 5 shows rainfall-runoff-sediment production relations for the rangeland watershed for 1967. Runoff from watersheds R-1 through R-4 has been low during the 5-year period of record. The 5-year (1963-1967) average annual runoff from these four watersheds, R-1, R-2, R-3, and R-4, was .06, .25, .11, and .22 inches, respectively. Precipitation was approximately equal to the long-term normal of 31.60 in 1967 and below normal in the other years of record. No high-intensity storms have occurred during the period of record.

Runoff amounts from grassland watersheds R-5 through R-8 were much greater than from R-1 through R-4 in 1967. Watersheds R-5 through R-8 were activated during 1966; therefore, only one full year of records is available. Precipitation in 1967 on these watersheds was below the long-term normal, with values ranging from 27.01 to 27.98 inches. Native grassland watersheds R-5 and R-6 reacted similarly with 1.16 inches and 1.23 inches of runoff, respectively. However, runoff on the formerly cultivated watersheds R-7 and R-8 was much larger, 3.89 inches and 3.05 inches, respectively. Severe erosion during and subsequent to cropping of watersheds R-7 and R-8 appear to be one reason for the greater runoff.

The severe soil loss for watersheds R-7 and R-8 reported in Table 5 was primarily from "inter-grass-clump" areas, indicating that these watersheds had not yet stabilized.

Cropland - Table 5 also shows runoff and sediment production for 1967 for cropland watersheds C-1 through C-8. These watersheds are bottomland, with slopes generally less than 0.1 percent. Farming operations on all watersheds are consistent with normal procedures within the area. In 1967, the runoff from watersheds C-3 and C-4 in irrigated cotton and C-7 in dryland row crops was 2.50, 1.39, and 1.91 inches, respectively. These watersheds normally have a large volume of runoff when compared with the other watersheds. Some of this runoff is from non-tilled, irrigation tailout ditches and turnrows. Runoff and soil loss from watersheds C-5 and C-6, sown to wheat and farmed alike were 1.32 and 1.19 inches respectively. Runoff and soil loss from watersheds C-1 (dryland row crop), C-2 (dryland row crop), and C-8 (dryland alfalfa) were very low compared with those from other watersheds.

Rainfall-runoff-sediment data from the cropland watersheds have been collected for only two years and no definite trends have been established.

Table 5.--Summary of rainfall, runoff and sediment loss from each of 16 unit source watersheds - 1967.

Watershed No.	Area (acres)	Rainfall (inches)	Runoff (inches)	Sediment loss (pounds)
-Rangeland-				
R-1	17.76	31.87	.09	1/
R-2	24.08	30.97	.44	1/
R-3	25.84	30.29	.16	1/
R-4	18.12	32.05	.42	1/
R-5	23.72	27.98	1.16	2,150
R-6	27.22	27.94	1.23	3,300
R-7	19.19	27.01	3.89	44,250
R-8	18.46	27.44	3.05	57,350
-Cropland-				
C-1	17.83	26.76	0.08	690
C-2	32.28	24.85	0.16	1/
C-3	44.26	25.60	2.50	111,150
C-4	29.33	25.55	1.39	36,100
C-5	12.75	24.30	1.32	3,400
C-6	13.00	24.66	1.19	3,800
C-7	26.52	24.38	1.91	31,800
C-8	27.28	24.19	0.57	250

1/ Not measured

Chickasha Pumping Sampler

Sediment yield from 11 of the 16 unit source watersheds was determined during 1967 by analysis of water samples hand collected during periods of runoff. Since runoff often starts within a few minutes after rain begins, it was impossible to be there early enough to obtain a sufficient number of samples from several watersheds at appropriate times to accurately calculate sediment quantities. To obtain a continuous curve of sediment concentration, frequent sampling during the early portion of a runoff event is required. Automatic sampling equipment is needed to do this economically. The only automatic samplers available were the large and expensive point samplers in use at larger stations. Technicians at Chickasha, therefore, designed and built a smaller and less expensive automatic pumping sampler suitable for these small watersheds. It is capable of obtaining a sufficient number of water samples to adequately show the shape of a sediment concentration curve for most storm events.

This new sampler is powered by a regular 12-volt D.C. automotive wet cell storage battery. A mercury switch on a float in the FW-1 gage well, located a few feet upstream from the weir, activates the system when flow commences over the weir. A 12-hour electric clock powers a notched time selector disc that controls the frequency of sampling. The present sampler is constructed to obtain samples at 10-minute intervals during the initial period of runoff, increasing by small intervals of time to 60 minutes during the final period of sampling. The sampling interval may be changed by changing the notched time interval selector disc. A rotating circular tray holds 28 pint sample bottles.

The electrical system consists primarily of a 12-volt battery, electric clock, a four-unit electric timer, and a system of micro switches and solenoids. As the time interval selector disc rotates, the arm of a micro switch slides into one of the notches on the disc. This turns the four-unit electric timer on, and the rotating tray is advanced to the fill position under the diverter discharge spout. A small self-priming electric pump then starts pumping water from the stream. The first portion of the water that is pumped purges the lines for about 30 seconds, then a solenoid positions the diverter to receive the sample. After the sample is obtained, the diverter is repositioned and pumping stops. A short tick mark is made on the FW-1 water level chart during the pumping operation by use of a solenoid that moves the FW-1 float tape. The sampling operation is complete. A second sample is taken when the unit receives another electrical impulse from the time selector disc. The operation is continued until 28 samples are taken, or the mercury switch breaks contact with the clock or is reset by a technician. A safety switch on the time selector disc prevents the refilling of bottles already filled. The unit functioned satisfactorily during eight runoff events sampled in 1967. Similar units installed in all unit source watersheds would insure obtaining sufficient data to calculate a fairly accurate estimate of sediment production.

SEDIMENT TRANSPORT AND CHANNEL STUDIES

Paul B. Allen and Norman H. Welch

The amounts of sediment carried by the river and its tributaries are measured at all runoff measuring stations. These sediment transport data and the channel research data are used to determine changes caused by upstream conservation works and to develop equations of channel behavior and sediment transport.

Sediment Yields

1. Short-term yields: Annual yields from 15 watersheds are shown in Table 6. Mean daily loads and incremental transport rates were used to determine the annual yields. The data in Table 6 were computed from 19,916 suspended sediment samples taken over the 6-year period. Of these samples, 2,130 were sieved and the particle size distribution was determined with a visual-accumulation-tube apparatus. Bed material gradation analyses were made on 705 bed samples.

2. Long-term yields: No analysis has been made of long-term yields. However, the long-term yields are in all probability greater than the 1962-67 short-term yields shown in Table 6. This period includes several drought years and lacks large out-of-bank flows.

Table 6.--Annual sediment transport for Washita River and tributary gaging stations, 1962-67.

Station	Drainage area (sq.mi.)	Apprx. pct. sand in meas. load	Thousands of tons					
			1962	1963	1964	1965	1966	1967
Washita River at Anadarko	3,656	9	1,300	172	574	951	73.4	92.5
Washita River at Verden	4,083	18	1,430	172	668	1,120	110	230
Washita River at Chickasha	4,328	13	1,470*	185*	591	1,200	136	284
Washita River at Tabler	4,707	15			1,050	1,780	265	478
Washita River at Alex	4,787	17	2,290	348	1,260	2,030	292	534
Tonkawa Creek	26.0	45		1.7	17.2	4.6	0.4	2.3
Sugar Creek	201.5	67			69.1	257	40.7	124
Delaware Creek	40.1	39			6.5	4.2	0.2	3.2
Line Creek	53.4			3.3	3.2	2.5	0.2	11.9
West Bitter Creek	60.8	9		64.5	13.6	44.9	43.3	65.7
East Bitter Creek	35.0	12			65.1	46.2	39.8	67.8
Bedingfield Creek	19.2					29.2	25.2	48.3
Little Washita R.	207.7	30			429	172	30.9	65.5
Big Dry Creek	7.6					1.2	2.2	0.2
Winter Creek	33.3	49			141	42.4	7.5	12.4

*Data from old station 5.7 miles upstream

3. Effect of flood-detention dams: Sediment yields were only one-tenth as large after dams were installed on Winter Creek. This was determined with a double-mass yield curve of data from Winter Creek and the adjacent, nontreated East Bitter Creek watershed. Such a large reduction is not expected in the long-term yield on Winter Creek because the analysis period was generally rainfall deficient. Reductions of this magnitude are not expected on all watersheds because the Winter Creek reduction included a decrease in channel erosion as well as storage of upland sediments in reservoirs.

Mechanics of Sediment Transport

1. Sediment and velocity distribution: To compare total sediment load computations made with the Brooks, modified Einstein, and Colby procedures, several sediment and velocity distribution studies were made in 1963 and 1964 on the Washita River. Tables 7 and 8 give a summary of the sediment and flow data for the studies.

Table 7.--Summary of sediment and velocity distribution data

Washita River Station	Disc. cfs	Vel. fps	Measured conc.- ppm		z				von Karman k	
					.062-.125 mm		.125-.250 mm			
			Total	Sand	Range	Avg.	Range	Avg.	Range	Avg.
Anadarko	1,630	4.49	5,580	600	.08-.27	.16	.17- .72	.45	.31-1.32	.63
Anadarko	3,860	4.53	5,300	820	.08-.26	.14	.25- .69	.40	.55-1.33	.95
Verden	4,360	3.82	4,890	880	.08-.28	.19	.13- .60	.39	.18-1.75	.63
Chickasha	2,390	3.48	10,700	590	.03-.20	.14	.17- .44	.35	.36-1.07	.60
Alex	1,250	3.13	8,800	1,580	.13-.83	.49	.41-1.13	.75	.20- .88	.43
Alex	4,250	4.26	7,490	2,010	.30-.73	.46	.46-1.41	.88	.15- .38	.25

Table 8.--Summary of total load data

Washita River Station	Disc. cfs	Slope ft/ft	Manning's n	Fr. no.	Bed D_{50} mm	Sand transport rates tons/day				Unmeas. load (by Brooks)	
						Meas.	Modified Einstein	Colby	Brooks	Pct. of sand load	Pct. of total load
Anadarko	1,630	0.0022*	.039	.40	**	2,460	--	3,200	2,770	11.2	1.3
Anadarko	3,860	0.00095	.041	.28	**	7,940	--	4,250	8,100	2.0	0.3
Verden	4,360	0.00036	.032	.22	0.200	10,400	12,200	13,800	10,500	1.0	0.2
Chickasha	2,390	0.00036	.033	.22	0.180	3,860	4,620	5,570	3,970	2.8	0.3
Alex	1,250	0.00032	.025	.24	0.140	5,290	6,310	7,750	7,140	26.0	5.9
Alex	4,250	0.00032	.021	.29	0.140	23,100	27,700	30,500	29,300	21.1	6.7

*Approximate slope for this rock control reach
**Solid sandstone bed

2. Comparison of total load procedures: Total sand transport rates computed by the modified Einstein, Colby, and Brooks procedures are shown in Table 8. The spatial distribution of silt and clay in the measured zone of the stream was quite good. For example, the silt and clay concentration varied only from about 4,200 ppm near the surface to 4,400 ppm 0.3 foot from the bed for the 3,860-cfs flow at the Anadarko station. From this it was assumed that the silt and clay concentration in the unmeasured zone was essentially the same as in the measured zone. Therefore, the measured load was used as the total load. Because the Brooks procedure uses field-determined z and k values, it is believed to provide the most reliable estimate even though it does not account for bed load. Calculations indicate that bed load constitutes less than 2 percent of the total sand load or less than 1/2 percent of the total load for the studied flows. This small percentage is due to the fine bed material, low velocities near the bed, and relatively narrow channel bed widths. For example, for the 4,250-cfs flow at the Alex station, if the thickness of the moving bed is taken as twice the mean particle size, the velocity of the bed movement assumed to be 1 fps, the width to be 100 feet, and the bed material volume weight to be 100 pounds per cubic foot, the bed load would be 0.7 percent of the total sand transport. Table 8 shows that the Washita River in the vicinity of Verden and Chickasha, Oklahoma carries virtually all transported material as suspended load. Where the channel is wider, as near Alex, however, roughly 25 percent of the sand load or 6 percent of the total load cannot be measured. It appears from the Table 8 data that both the Colby mean velocity method and modified Einstein total load procedures are satisfactory for computing total load in the lower reach. Because each Brooks computation requires lengthy field, laboratory, and office work, the Brooks method is used only occasionally to check the accuracy of other methods.

3. Effect of flow temperature on transport: For 3 days during the September 1965 bank-full flow on the Washita River near Alex, a positive correlation existed between flow temperature and the fine material in transport. This was believed to be the result of increased bank erosion caused by an increase in turbulence with warm water. This positive correlation between temperature and transport of fine particles is especially interesting, because other researchers have found numerous instances of negative correlation between temperature and the transport of coarse material.

Computational Procedures

1. Continuous sediment concentration method: The data in Table 6 were computed by taking sufficient suspended sediment samples during runoff events to construct continuous concentration curves on the stage recorder charts. Measured transport was determined by a chart reader and computer using the relation

$$T = \Sigma\, k \Big[(C_1 + C_2)(Q_1 + Q_2)(t_2 - t_1)/4 \; . \; . \; . \\ + (C_{n-1} + C_n)(Q_{n-1} + Q_n)(t_n - t_{n-1})/4 \Big]$$

where

T = transport in tons
k = units conversion constant = 0.0001123
C = concentration in ppm
Q = discharge in cfs
t = time in hours

2. Unmeasured load: For each gaging station with significant unmeasured load, the modified Einstein total load procedure was used to develop a discharge vs. unmeasured load curve. Unmeasured load determined from these curves was added to the measured load to obtain the total load data shown in Table 6.

3. Sediment rating curve vs. continuous concentration curve methods: A comparison is being made of sediment discharge from small watersheds (30 to 50 sq. mi.) as computed by these two methods. Agreement of the estimates by the two methods was erratic when only one year of flow and sediment data was used. Differences ranged from 8 to 100 percent. Agreement was very good, however, when two years of flow and sediment data were used. Differences then ranged from 1 to 4 percent. Good sample coverage is especially important during the higher stages when most of the transport takes place. Essentially the same flow duration curve resulted when mean daily flows were used with the instantaneous flow peak plotted at the 0.01 percent time point as when time-incremental (down to 6 minutes) flow data were used. Accurate definition of flow duration curves is needed at the high-flow end because over half the sediment can be transported between the one-day frequency (0.27 percent frequency) and the 0.01 percent frequency.

Channel Bed Studies

1. Particle-size distribution of bed vs. suspended material: Because the gradation of the bed material is one parameter used in computing unmeasured load, an understanding of bed material changes is desirable. In 1964, bed samples were taken throughout a runoff event at Verden that peaked at 6,300 cfs. The gradation of samples taken with a BM-54 sampler tended to follow the gradation of the suspended material, suggesting a continual exchange of particles between the bed and the suspended load. The bed contained more silt and clay when the flow was near the peak because these particles would get trapped in the churning bed. During and after the recession, these silt and clay particles were gradually washed from the bed.

2. Gradation of bed with depth: Short reaches of the Washita River bed have been sampled intensively in cross section and depth (5'-8') at Verden, Chickasha, and Pauls Valley. The median particle size varied in each section from less than 0.2 mm near the surface to greater than 0.5 mm 4 to 8 feet below the surface in one or more areas. The cause of these coarse areas is not understood, but sub-bed flow is suspected to influence their development.

Channel Behavior Studies

1. Aggradation or degradation: During the past 6 years numerous flow events with associated channel scour or fill, or flood plain scour or fill have been observed or recorded. Generally, channels and flood plains have changed little in watersheds with principally shale geology. However, large channel and flood plain changes have occurred in sandstone geology watersheds. Intense spring thunderstorms that produce very high sediment concentrations and less than bank-full flows have caused the most channel aggradation. Slightly out-of-bank flows on these sandy tributaries cause considerable over-bank deposition. The extremely large 8,600-cfs flow on Sugar Creek in September 1965 caused channel bed scour, large sand splays along the banks, and flood plain scour at various places in the valley.

2. Channel shape vs. material in transport: Streams with a high sand load are wide and shallow. The width-to-depth ratio (w/d) is defined by the equation, $w/d = 0.17X + 3.55$ where X is the percent sand in the measured load.

3. Sediment progression: The intensive sample collection during storm flow at the sampling stations has expanded our understanding of the progression of sediment through the basin. Continuous sediment concentration curves of large flows that originate entirely above Anadarko retain almost the identical shape through the stable channel past Chickasha. Below Chickasha, where the Washita River channel is wider and less stable, there is a fairly large pickup in sediment during large flows. This is caused by the flushing action of the large flows on sediment stored in the channel. The stored sediment results when large tributary flows enter the Washita River when the river is at low flow and deposit much of their load. Cross sections surveyed by the Corps of Engineers, U. S. Bureau of Reclamation, and Agricultural Research Service have shown that in the Tabler-Alex reach of the Washita River the bed level has fluctuated as much as 6 feet in a short period of years because of the storing and flushing actions.

4. Bank erosion: Evidence is accumulating that excessive sand loads contribute to accelerated bank erosion on the Washita River. Although the mechanics of this erosion are not understood, consideration might still be given to sand transport regulation in a major bank-protection plan.

5. Channel shape vs. vegetation: A vegetation species and population count was made at several points on the Washita River to see if the type of vegetation affects channel shapes. The results were inconclusive; however, a plant succession was observed. As alluvial deposits age and build higher above the water table, the woody plant sequence is salt cedar, willow-cottonwood, cottonwood, and cottonwood-elm-hackberry.

6. Cross sections: To document channel changes, approximately 200 channel cross sections are being surveyed periodically on the Washita River and tributaries.

Pumping Samplers

1. XPS-62 pumping-type sediment sampler: One sampler of this type has been used for two years at two Washita River stations. After initial malfunctions were corrected, the sampler has operated correctly and has proved to be mechanically, hydraulically, and electrically dependable. The greatest problem remaining is finding a location for the intake so that pumped sample concentration will have a consistent relation with the average suspended concentration of the stream.

2. PS-66 pumping-type sediment sampler: Two of these were installed on the Washita River in 1967. Neither collected representative samples with the original 2-minute pumping cycle timer. No samples have been collected since the 2-minute timers were replaced with 5-minute timers.

3. Chickasha pumping sampler: These 28-bottle, small, fully automatic samplers are built locally for small watersheds. They are described under unit source watershed runoff studies.

GEOMORPHOLOGIC STUDY OF THE WASHITA RIVER BASIN

Don W. Goss, Alex R. Ross, and Tommy B. Thompson

A knowledge of the past history of the Washita River is very important in understanding and predicting future trends. To obtain the required geomorphologic history and geomorphic form of the river, several lines of study have been undertaken recently. Preliminary results of these studies will be discussed briefly.

To determine changes in alignment and length of the Washita River channel, a map was compiled from aerial photographs showing the location of the channel in 1937, 1948, 1955, and 1961. In addition, the channel locations obtained from original land surveys were added to the map. Channel lengths for each year for which data are available have been measured on the map and the results summarized in Table 9. To date, no maps have been located showing the Washita River channel accurately in the interval between the original 1873 survey and the earliest aerial photographs of 1937.

Table 9.--Channel length changes, Washita River, 1873-1961

Year	A	B	C
	- - - - - - - - - - Miles - - - - - - - - - -		
1873	42.3	----	----
1898	----	23.7	14.0
1937	37.9	25.2	18.5
1948	38.0	25.4	17.5
1955	35.3	25.6	14.1
1961	36.5	25.7	13.6

Column A is distance from Anadarko gage site to a point on the river 2 miles west of Chickasha. This is the area covered in the 1873 land survey.

Column B is distance from end point of 1873 survey to the mouth of the Little Washita River.

Column C is distance from the mouth of the Little Washita River to the Alex gage site.

Dating of the alluvial deposits is very important in the development of the geomorphic history of the valley. Animal bones recovered from flood-plain and terrace deposits have been identified by personnel of the Museum of the Great Plains at Lawton, Oklahoma as modern cattle, canine, and bison. It was evident from the manner of occurrence of the bones that they represented burial <u>in-situ</u> shortly after death. A group of bison bones sent to the USDA Sedimentation Laboratory, Oxford, Mississippi for carbon dating were less than 200 years old.

Two buried soils from lower terrace deposits of the Washita River were dated by L. L. McDowell at the USDA Laboratory at Oxford, Mississippi. The deepest soil, 10 feet below the terrace surface, was dated as 1,760 ± 150 years Before Present (B.P.). The shallowest soil, 8 feet below the terrace surface, was dated as 1,000 ± 100 years B.P.

Charcoal samples collected 7 and 9 feet below the surface of the alluvium were dated as 2,025 ± 150 and 2,215 ± 150 years B.P. respectively. These samples were collected in the Winter Creek channel bank, about 3 miles from its confluence with the Washita River.

A sample of wood was recently recovered from a depth of about 50 feet during drilling operations near Anadarko. Samples of this wood were dated by Isotopes, Incorporated as 3,850 ± 110 years B.P.

Mr. A. D. Bull, Soil Conservation Service (retired), Chickasha, collected some walnut from Sandstone Creek (Roger Mills County) at a depth of 13.7 feet and some Bois d'Arc from Criner Creek (McClain County) at a depth of 8 feet. These samples were dated as 1,025 ± 100 and 1,548 ± 75 years B.P. respectively. Both Sandstone Creek and Criner Creek are tributaries of the Washita River.

A mammoth has been uncovered in the headwaters of Tonkawa Creek. The excavation of the mammoth was directed by the Museum of the Great Plains, Lawton, Oklahoma, and has been termed the Domebo site. From radiocarbon datings of various organic remains at the Domebo site, the age of the site was concluded to be about 11,200 years B.P. Table 10 shows the known radiocarbon dates of materials from the Washita River Basin.

Table 10.--Radiocarbon dates from the Washita River Basin

Material	Location	Depth in feet	Age Years B.P.
Bison bones	Washita River bank	14	Recent
Bison bones	Washita River bank	4	Recent
Bison bones	Washita River bank	--	<200
Buried soil	NW 1/4 SE 1/4 Sec. 20, T. 6N.,R. 6W., Grady Co.	8	1,000 ± 100
Buried soil	NW 1/4 SE 1/4 Sec. 20, T. 6N.,R. 6W., Grady Co.	10	1,760 ± 150
Charcoal	SE 1/4 NE 1/4 Sec. 25 T. 6N.,R. 6W., Grady Co.	7	2,025 ± 150
Charcoal	SE 1/4 NE 1/4 Sec. 25 T. 6N.,R. 6W., Grady Co.	9	2,215 ± 150
Walnut	Sandstone Creek	13.7	1,025 ± 100
Bois d'Arc	Criner Creek	8	1,548 ± 75
Mammoth (Domebo site)	Tonkawa Creek	Streambed	11,200
Wood	Terrace near Anadarko	50	3,850 ± 110

Two projectile points have been uncovered from alluvial deposits in the Washita River terrace. The first was discovered in the bank of the Washita River, and identified by Dr. R. E. Bell of the University of Oklahoma Museum as a Washita type in use from 1400-1500 A.D. Unfortunately, it cannot be shown that this represents the original burial of the point. The second projectile point was uncovered during coring operations at a depth of 73 feet. The age of this point was placed by Dr. Bell at about 7,000 years.

In order to describe the physical characteristics of the Spring Creek watershed, a number of geomorphic parameters have been determined. This was in conjunction with a subdivided watershed study. After runoff data are obtained from the watersheds, the relationship, if any, of these geomorphic parameters to hydrologic performance will be determined. The parameters chosen are those that have been accepted by geomorphologists to be definitive or indicative of some character of the watershed. These parameters are area, total stream miles, drainage density, stream orders, number and length of streams in a given order, basin perimeter, and maximum basin length parallel to the major drainage line. Also, the circularity ratio and the elongation ratio, which are dimensionless parameters, have been calculated. The parameters for

Spring Creek watershed are presented in Table 11. Most of these parameters have also been determined for the basins of stream order three or greater in the Little Washita River watershed above Charlie Creek. These are presented in Table 12.

Dimensionless parameters are quite useful in defining watersheds. Many times they become valuable tools in the development of mathematical relations pertaining to hydrologic characteristics of the basin. New dimensionless parameters have been investigated and, thus far, two have been developed that warrant additional study. They have been defined as the length-width ratio and the basin eccentricity. Both of these parameters are derived from an equivalent elliptical form of the basin. Only the area (A) and the maximum length of the basin parallel to the main drainage line (L) are required to determine this equivalent form. An ellipse is defined by the equation

$$\frac{X^2}{a^2} + \frac{Y^2}{b^2} = 1$$

In this relation "a" is one-half the length of the major axis and "b" is one-half the length of the minor axis of the ellipse. The perimeter of the ellipse is defined by "X" and "Y" values satisfying the equation. The dimensions of the equivalent ellipse for the basin are determined by assigning to "a" the value $\frac{L}{2}$ and by calculating "b" from the relation $A = \pi ab$, where A is the known area of the basin. The ellipses constructed to scale of four selected third-order basins of the Little Washita drainage system are represented in Figure 15. Figures 16 and 17 show the comparison of the elliptical form to the actual shape of each basin. It can be seen that the elliptical form is very representative of the true basin shape.

The shape of an ellipse can be defined in a comparative form by using the two previously mentioned dimensionless parameters. The length-width ratio (W) is the length of the major axis divided by the length of the minor axis.

$$W = \frac{a}{b}$$

This value will be one for a circle, and approach infinity as the ellipse becomes so narrow that it appears to be a straight line. The shape of an ellipse is also dependent on its eccentricity (E).

$$E = \frac{b}{a} \left[\left(\frac{a}{b}\right)^2 - 1 \right]^{\frac{1}{2}}$$

Table 11.--Spring Creek watershed geomorphic parameters

Stream gaging sites	Area (miles^2)	Total stream length (miles)	Drainage density (miles/miles^2)	Stream orders 1/ (number in each)				Total length for each order Mean length for each order (miles)				Circularity ratio 2/	Elongation ratio 3/	Basin perimeter (miles)	Maximum length of basin (miles)	Mean bifurcation ratio
				1st	2nd	3rd	4th	1st	2nd	3rd	4th					
No. 1 (5141)	6.447	37.06	5.75	125	34	10	1	13.62 0.149	8.04 0.236	7.620 0.762	2.72 2.72	.618	.774	12.541	3.705	3.94
No. 2 (5142)	0.575	3.46	6.17	11	4	1	0	2.21 0.201	0.74 0.185	0.510 0.510	-- --	.887	.841	3.033	1.016	3.06
No. 3 (5143)	0.803	3.78	4.71	15	3	1	0	1.56 0.104	1.57 0.523	0.670 0.670	-- --	.777	.618	4.098	1.639	4.64
No. 4 (5144)	1.577	10.59	6.71	34	11	1	0	5.96 0.175	2.30 0.209	2.330 2.330	-- --	.772	.636	5.770	2.229	3.96
No. 5 (5145)	0.395	2.71	6.86	13	2	1	0	1.56 0.120	1.10 0.55	0.050 0.050	-- --	.397	.600	3.082	1.180	5.75
No. 6 (5146)	0.827	4.96	6.00	16	4	1	0	2.78 0.174	1.19 0.297	0.980 0.980	-- --	.337	.763	3.852	1.344	4.10

1/ Stream orders follow Strahler's method rather than Horton's.
2/ Circularity ratio - ratio of circumference of a circle of same area as basin to the basin perimeter.
3/ Elongation ratio - ratio of diameter of a circle with same area as the basin to the maximum length of basin.

Table 12.--Geomorphic parameters for certain basins in the Little Washita River watershed above Charlie Creek

Basin identification number	Area (miles^2)	Number of streams in each order 1/ 1st	2nd	3rd	4th	Circularity ratio	Elongation ratio	Basin perimeter (miles)	Maximum basin length (miles)	Mean bifurcation ratio
$3_1 4_1 5_1$	4.293	17	4	1		.6803	.5768	10.796	4.053	4.202
$3_2 4_1 5_1$	2.627	9	2	1		.7778	.5487	7.387	3.333	3.964
$4_1 5_1$	7.287	26	6	2	1	.7773	.6757	12.311	4.508	3.922
$3_3 4_2 5_1$	1.205	5	2	1		.7338	.6540	5.303	1.894	2.350
$3_4 4_2 5_1$	4.444	17	3	1		.7174	.6978	10.417	3.409	5.221
$4_2 5_1$	5.804	23	5	2	1	.6710	.7716	12.728	3.523	4.008
$3_5 5_1$	5.511	13	4	1		.7524	.6189	11.061	4.280	3.420
$3_6 5_1$	2.864	15	3	1		.8163	.6303	7.349	3.030	4.636
$3_7 4_3 5_1$	3.626	13	2	1		.6552	.5064	10.303	4.243	5.750
$3_8 4_3 5_1$	6.103	28	7	1		.6964	.5888	12.576	4.734	4.558
$4_3 5_1$	9.926	41	9	2	1	.7262	.6777	15.379	5.246	4.426
$3_9 4_4 5_1$	2.696	17	4	1		.8397	.7192	6.932	2.576	4.202
$3_{10} 4_4 5_1$	1.230	8	2	1		.8438	.9437	4.659	1.326	3.538
$4_4 5_1$	5.071	29	7	2	1	.7863	.6952	10.152	3.655	3.888
$3_{11} 5_1$	1.942	10	3	1		.7126	.6193	6.932	2.538	3.255
$3_{12} 5_1$	2.894	15	4	1		.8206	.7919	7.349	2.424	3.802
$3_{13} 4_5 5_1$	2.693	14	3	1		.7916	.6562	7.349	2.822	4.349
$3_{14} 4_5 5_1$	1.373	11	3	1		.8122	.7349	5.114	1.799	3.518
$3_{15} 4_5 5_1$	.433	4	2	1		.8102	.6818	2.879	1.089	2.000
$4_5 5_1$	7.560	43	10	3	1	.6482	.5243	15.038	5.918	4.046
$3_{16} 5_1$	2.306	12	2	1		.6865	.5026	7.841	3.409	5.294

Table 12.--Geomorphic parameters for certain basins in the Little Washita River watershed above Charlie Creek (cont'd)

Basin identification number	Area (miles^2)	Number of streams in each order 1/				Circularity ratio	Elongation ratio	Basin perimeter (miles)	Maximum basin length (miles)	Mean bifurcation ratio
		1st	2nd	3rd	4th					
$3_{17}4_{6}5_{1}$	2.262	10	3	1		.8328	.7529	6.402	2.254	3.255
$3_{18}4_{6}5_{1}$	1.487	11	3	1		.7409	.5526	5.834	2.490	3.518
$4_{6}5_{1}$	4.351	22	6	2	1	.8202	.7742	9.015	3.040	3.402
$3_{19}5_{1}$	5.465	26	2	1		.7570	.6633	10.947	3.977	11.935
$3_{20}4_{7}5_{1}$	2.209	14	4	1		.6954	.6554	7.576	2.557	3.609
$3_{21}4_{7}5_{1}$	1.444	8	3	1		.7497	.7232	5.682	1.875	2.756
$3_{22}4_{7}5_{1}$	1.201	7	2	1		.8691	.7504	4.470	1.648	3.125
$3_{23}4_{7}5_{1}$	.874	6	2	1		.8411	.8385	3.940	1.258	2.727
$3_{24}4_{7}5_{1}$	1.132	5	2	1		.9928	.9753	3.799	1.231	2.350
$4_{7}5_{1}$	12.884	88	24	5	1	.6786	.5306	18.751	7.633	3.945
5_{1}	94.624	438	104	24	7	.6949	.8800	49.623	14.072	4.231

1/ Following Strahler's method

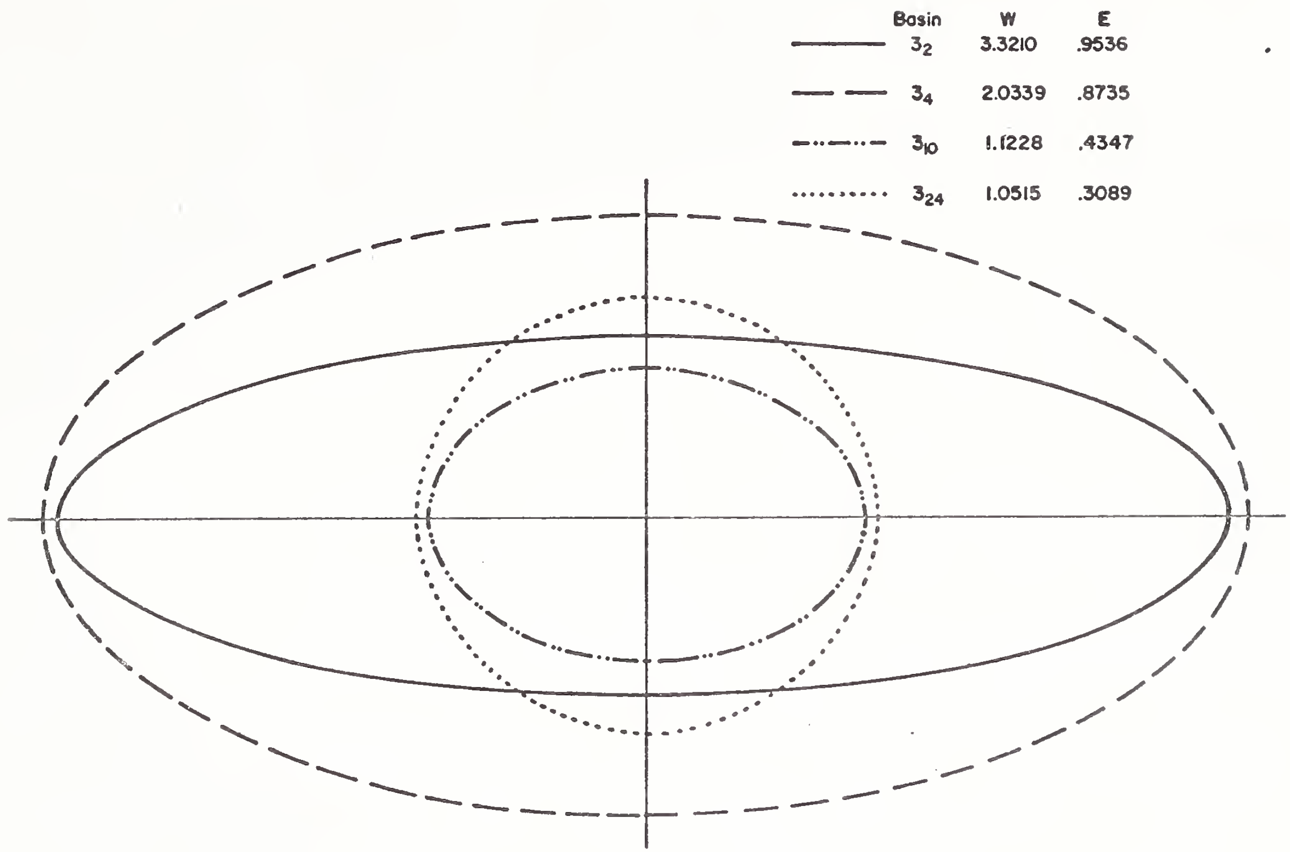

Figure 15
Ellipses constructed to scale of four selected basins

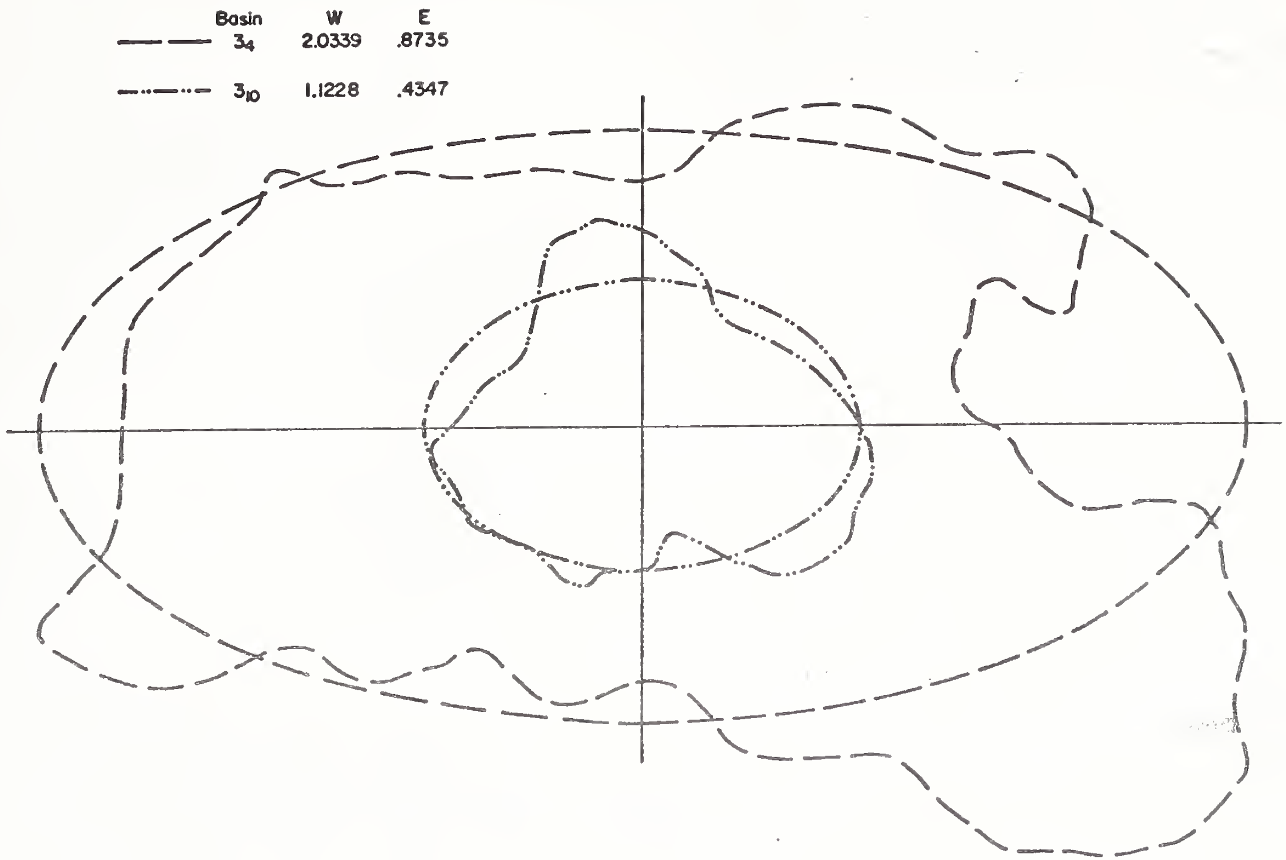

Figure 16
Comparison of the elliptical form of two basins to the actual basin shapes

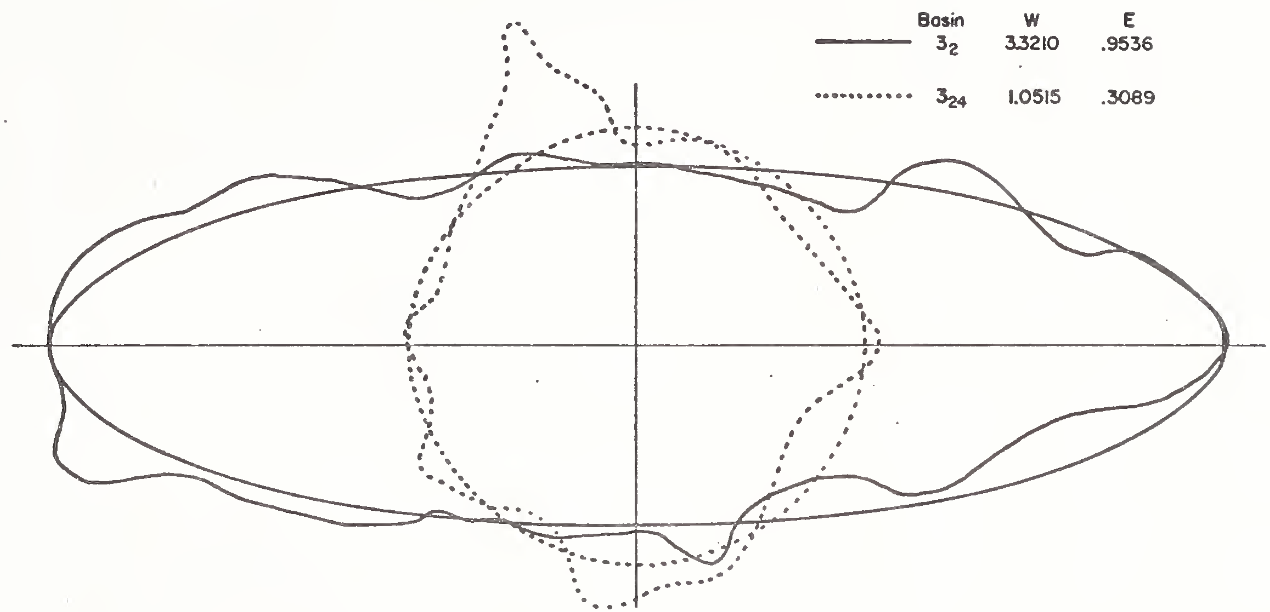

Figure 17
Comparison of the elliptical form of two basins to the actual basin shapes

This value will be zero for a circle, and approach one as the ellipse approaches a straight line. Figure 18 shows the elliptical forms of the four basins reduced to a common minor axis so that a direct comparison may be made of their shape.

Table 13 compares the length-width ratio and eccentricity to the previously established elongation ratio and circularity ratio for Spring Creek and Little Washita River watersheds. Later, these dimensionless parameters will be related to measured hydrologic characteristics of the basins.

In addition to the previously mentioned geomorphic parameters for Spring Creek, hypsometric integrals--the percent mass of a basin above the contour encompassing 50 percent of the area--have been computed. Table 14 lists this parameter for Spring Creek.

Table 14.--Hypsometric integrals for basins of Spring Creek watershed

Basin identification number	Hypsometric integral
1	.46
2	.62
3	.60
4	.54
5	.54
6	.54

Dr. T. A. Bond studied pollen from oxbow deposits of the Washita River as a portion of his dissertation study, University of Oklahoma. The objectives of the investigation were to determine the nature, successional history, environmental significance and relative ages of oxbow lake deposits in central and southeastern Oklahoma.

It was concluded that spores and pollen are useful when used in conjunction with carbon 14 in dating sediments of oxbow lakes. When the pollen and spore assemblages from the oxbow lakes were compared with those of the Domebo site it appeared that the oxbows are older than 7,000 but younger than 10,123 years B.P. This would place them in the late Wisconsin age. An oxbow lake sampled near Chickasha was thought to be older than either of two oxbow lakes sampled near Anadarko, because an assemblage of spores and pollen indicative of an earlier period in the climatic succession were found in the Chickasha oxbow.

Observation wells have been drilled along the valley in the study reach on 1-mile centers or less, except for the Sugar Creek area. These wells have provided enough information so that an isopachous map of river fill has been completed except for T. 7N, R. 10W (Anadarko), and the Sugar Creek area. The volume of unconsolidated sediment without the two areas mentioned above is 1.26 cubic miles over an area of 127 square miles. The average thickness of fill over this area is 52.1 feet. Once the volume of sedimentary fill is known, the volume of ground water in storage can be calculated monthly and compared directly with precipitation and other upstream variables.

Table 13.--Dimensionless geomorphic parameters for Spring Creek and Little Washita watersheds

Basin identification number	Circularity ratio	Elongation ratio	Length-width ratio	Eccentricity
Spring Creek				
1	.618	.774	1.6723	.8015
2	.887	.841	1.4100	.7050
3	.777	.681	2.6275	.9247
4	.772	.636	2.4745	.9147
5	.397	.600	2.7687	.9325
6	.837	.763	1.7155	.8125
Little Washita				
$3_1 4_1 5_1$	.6803	.5768	3.0053	.9430
$3_2 4_1 5_1$	.7778	.5487	3.3210	.9536
$4_1 5_1$	.7773	.6757	2.1905	.8897
$3_3 4_2 5_1$	.7338	.6540	2.3383	.9039
$3_4 4_2 5_1$	.7174	.6978	2.0539	.8735
$4_2 5_1$	.6710	.7716	1.6795	.8034
$3_5 5_1$	.7524	.6189	2.6107	.9237
$3_6 5_1$	.8163	.6303	2.5179	.9177
$3_7 4_3 5_1$	.6552	.5064	3.8998	.9666
$3_8 4_3 5_1$	.6964	.5888	2.8841	.9379
$4_3 5_1$	.7262	.6777	2.1777	.8833
$3_9 4_4 5_1$	.8397	.7192	1.9331	.8558
$3_{10} 4_4 5_1$	.8438	.9437	1.1228	.4547
$4_4 5_1$	.7863	.6952	2.0692	.8755

Table 13.--Dimensionless geomorphic parameters for Spring Creek and Little Washita watersheds (cont'd)

Basin identification number	Circularity ratio	Elongation ratio	Length-width ratio	Eccentricity
Little Washita				
$3_{11}5_1$	.7126	.6193	2.6052	.9234
$3_{12}5_1$	.8206	.7919	1.5947	.7790
$3_{13}4_5 5_1$	.7916	.6562	2.3226	.9026
$3_{14}4_5 5_1$	.8122	.7349	1.8515	.8416
$3_{15}4_5 5_1$	.8102	.6818	2.1513	.8854
$4_5 5_1$	.6482	.5243	3.6387	.9615
$3_{16}5_1$	.6865	.5026	3.9584	.9676
$3_{17}4_6 5_1$	.8328	.7529	1.7642	.8238
$3_{18}4_6 5_1$	.7409	.5526	3.2746	.9522
$4_6 5_1$	.8202	.7742	1.6683	.8004
$3_{19}5_1$	.7570	.6633	2.2731	.8980
$3_{20}4_7 5_1$	.6954	.6559	2.3612	.9059
$3_{21}4_7 5_1$	.7497	.7232	1.9121	.8523
$3_{22}4_7 5_1$	.8691	.7504	1.8546	.8422
$3_{23}4_7 5_1$	.8411	.8385	1.4221	.7110
$3_{24}4_7 5_1$	.9928	.9753	1.0515	.3089
$4_7 5_1$	.6786	.5306	3.5519	.9595
5_1	.6949	.8300	1.6437	.7936

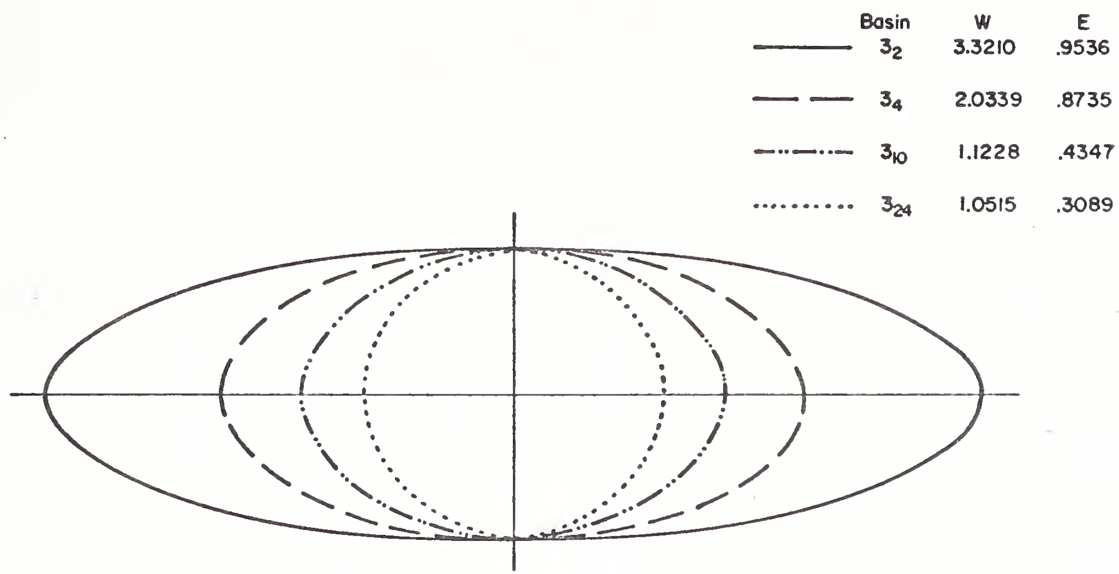

Figure 18
Ellipses of the four selected basins reduced to a common minor axis to compare their shapes

CALIBRATION OF TRIANGULAR WEIRS

Bruce J. Blanchard

Some measuring stations with weir controls being used at Chickasha on the subdivided watershed studies are in remote areas and are difficult to reach in time to calibrate with discharge measurements in the field. A reliable prediction equation for discharge, along with a few discharge measurements to use as checks, would eliminate the necessity of calibration by modeling. The standard form of the equation for discharge over a sharp-crested, V-notch weir, $Q = CH^{2.5}$, is inadequate when effects from variable approach velocities, submergence, and fast-rising flows are encountered. A comprehensive analysis of the stage-discharge relationships of a triangular weir was undertaken to improve the prediction of runoff from stream-gaging stations with weir controls.

A theoretical approach to flow over a sharp-crested, triangular weir leads to the following equation:

$$q = \frac{8}{15} C_c Z \sqrt{2g} \left[\left[\alpha\frac{V_o^2}{2g} + h \right]^{\frac{5}{2}} - \left[\alpha\frac{V_o^2}{2g} \right]^{\frac{5}{2}} - \frac{5}{2} h \left[\alpha\frac{V_o^2}{2g} \right]^{\frac{3}{2}} \right] \tag{1}$$

where q = discharge, C_c = coefficient of contraction, Z = side slope of the weir, α = correction for nonuniformity of the approach velocity, V_o = mean approach velocity, and h = the head causing flow over the weir.

This basic equation does not take into account the modification of flow due to submergence, rate of change of stage or malformation in construction of the weir. The effects of submergence on discharge over a V-notch weir have been investigated in several laboratories, but all experiments were on weirs less than 3 feet wide. The weirs in use at Chickasha range up to 100 feet wide, and it was not known if the submergence relationships obtained in the laboratory experiments would hold true.

Corrections for rate of change of stage had been used for some time in computing flow of large rivers; however, suitable coefficients for use on streams in this area had not been determined. Corrections for these effects are made by multiplying the basic equation (1) by the submergence factor equation (2), and the rate of change of stage equation (3).

The effect of submergence on the discharge rate of a weir was assumed to be one of superposition in which actual flow was a function of the ratio of free flow downstream to free flow upstream under the

tailwater head. The submergence factor which is multiplied by the free flow discharge has the following form:

$$k\left[1.0 - \frac{C_{c_2}\, h_2^{\frac{5}{2}}}{C_{c_1}\left[\left[\alpha\frac{V_1^2}{2g} + h_1\right]^{\frac{5}{2}} - \left[\alpha\frac{V_1^2}{2g}\right]^{\frac{5}{2}} - \frac{5}{2}\, h_1\left[\alpha\frac{V_1^2}{2g}\right]^{\frac{3}{2}}\right]}\right]^m \qquad (2)$$

where k is a coefficient usually found to be 1.0, subscripts 1 and 2 refer to upstream and downstream variables respectively, and m is a constant. When h_2 is zero, the submergence factor is 1.0. Analysis of the data and the equations shows that k should be 1.0 and that α could not be predicted without plotting discharge measurements. The k was then dropped from the equation and a value of 1.08 was assigned to α because it gave the best fit of predictions.

Rate of change of stage over the weir was assumed to modify the flow rate by the following factor

$$\sqrt{1.0 + K\frac{dh}{dt}} \qquad (3)$$

which is also multiplied times the flow and in which $\frac{dh}{dt}$ is the rate of change of stage in feet per minute and K is considered constant[11].

In this study the discharge data from which the final equations were developed came from an actual weir which was slightly malformed. A correction, M, for this malformation was calculated as the ratio of theoretical discharge of a true-shaped weir to the theoretical discharge of the malformed weir. The measured discharge was multiplied by this factor to obtain discharges for a true weir with which a prediction equation could be developed. Therefore, to predict flow of any actual weir, the prediction equation must be multiplied by the reciprocal of corrections obtained in this same manner.

Coefficients in the above equations were evaluated by using 82 discharge measurements on one weir. The equations were then fitted for two different conditions; with and without velocity data. The resulting prediction equations are:

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[11]Corbett, D. M., and others, 1943 Stream Gaging Procedure - U. S. Geological Survey Water Supply Paper 888, page 162, (Reprinted 1945)

Without velocity data

$$q_o = 2.73\ Z\ h_1^{2.73} \left[1.0 - \left[\frac{h_2}{h_1} \right]^{2.73} \right]^{.405} \sqrt{1 + 2.13\frac{dh}{dt}(\frac{1}{M})} \qquad (4)$$

With discharge measurement data

$$q_o = 2.27\ Z\ h_1^{.214} \left[h + 1.08\frac{V_1^2}{2g} \right]^{2.5}$$

$$\left[1.0 - \frac{h_2^{2.71}}{h_1^{.214} \left[h_1 + 1.08\frac{V_1^2}{2g} \right]^{2.5}} \right] \sqrt{1.0 + .692\frac{dh}{dt}(\frac{1}{M})} \qquad (5)$$

where q_o = observed discharge in cfs, Z = least squares fitted weir side slope ratio - horizontal to vertical, h_1 = upstream head in feet above the crest adjusted for malformation, V_1 = mean upstream velocity in feet per second, h_2 = downstream head in feet above the crest adjusted for malformation, $\frac{dh}{dt}$ = rate of change of stage in feet per minute, and M = malformation coefficient.

The two most conventional equations:

$$Q = CH^n \qquad (6)$$

$$Q = CH^n \sqrt{1 + K\frac{dh}{dt}} \qquad (7)$$

were fitted to the data and comparison of the standard errors indicated that use of the proposed forms of the discharge equations (4) and (5) are justified.

EXPLORATORY STUDY OF THE REGIME OF WASHITA RIVER MAIN STEM FLOWS

Donn G. DeCoursey

Objectives of this study were to reproduce a 10-year regimen of flows on the main stem of the Washita River, then to calculate the effects upon this flow regimen of structural measures (primarily floodwater-retarding reservoirs) being installed in the Washita River Basin by the U. S. Department of Agriculture, as authorized by The Flood Control Act of 1944 and subsequent amendments[12]. To accomplish these objectives, synthetic floods of surface runoff on tributaries to the Washita River were developed then routed down the main channel of the Washita to form the regime of main stem flows. The unit hydrograph approach, although crude, was selected as the main tool for developing the tributary inflow. Records from gaging stations on tributaries to the main stem were used to derive synthetic unit hydrographs for all tributaries to the Washita River. Six of the gaged watersheds (Barnitz, Rush, Rock, Little Washita, Pond, and Sugar) provided a representative sample of the entire Washita watershed and had records suitable for hydrograph analysis. Changes in the flow regime at six main stem river stations (Cheyenne, Clinton, Carnegie, Tabler, Pauls Valley, and Durwood) were observed. The period, 1941-50 was selected because data was available for the entire period at all main stem river stations and there were no floodwater-retarding structures present. Figure 19 shows the location of the tributary and main stem gaging stations.

The following steps were completed in reaching the objectives:

1. Precipitation, runoff, and topographic data were collected.

2. A modified form of the Gamma function was used to define the shape of the tributary unit hydrographs[13].

3. The unit hydrograph peak rate, time to peak, and recession constant used in the hydrograph equation have been related to watershed characteristics as follows:

$$t_p = 0.085\ W^{2.15}$$
$$q_p = 46.1\ t_p^{-1.28}\ W^{2.55}\ l_{cg}^{1.08}$$
$$d = 1.76 \ln DA - 3.85$$

where t_p is the time to peak in hours, q_p is the peak rate in cfs, d is the recession constant, W is the average watershed width in miles, l_{cg} is the length of the watershed to the point opposite the centroid in miles, and DA is the drainage area in square miles. The value of t_p was rounded off to the nearest hour before being used in the prediction equation for q_p.

[12] In this paper these structural measures will hereinafter be referred to as the FRS Program.

[13] ARS 41-116.

Figure 19
Washita River sub-watersheds

4. The equation used to calculate the tributary runoff volume in inches was:

$$Q = M \cdot P \cdot I \left[API + .068\ P^{1.24} \right]$$

Q is the runoff in inches, M is a geologic constant, P is the precipitation, I is the maximum 1-hour rainfall intensity, and API is an antecedent basin moisture index which is a variable depending upon season. The depletion rate in the index is a function of the inverse of the long-term temperature curve and varies continuously from 0.67 in July-August to 0.90 in January.

Channel losses were assumed to vary from reach to reach, but were assumed to be constant within a reach. They were taken as a constant percent of the flow rate.

5. The Washita Basin was subdivided into 70 subwatersheds, the location and physical characteristics of which are shown in Figure 19 and Table 15, respectively.

6. The successive average-lag method of flood routing was used to route all unit hydrographs including channel losses to the main stem stations. Flood hydrographs were developed by summing the products of the routed unit hydrographs and the precipitation excess.

7. The effect of structures on local or tributary flow was determined by breaking the flow into two parts, uncontrolled flow from the drainage area below structures, and the outflow from structures. The unit hydrograph from the uncontrolled area is a modification of the original unit hydrograph for the tributary as follows:

$$t_u = 1.04\ t_t \left[\frac{DA_u}{DA_t} \right]^{0.159}$$

$$q_u = 1.12\ q_t \left[\frac{DA_u}{DA_t} \right]^{1.33}$$

where t_u and t_t are the times to peak of the uncontrolled area and the original tributary unit hydrographs respectively, q_u and q_t are the peak rates of the uncontrolled area and the original tributary unit hydrographs respectively, and DA_u and DA_t are the drainage areas of the uncontrolled area and original tributaries respectively.

Using these equations, unit hydrographs for the uncontrolled area were developed and routed to the main stem stations.

Table 15 Characteristics of subwatersheds in the Washita River Basin

Water-shed No.	Name	Watershed Characteristics: Total Drainage Area (sq.Mi.)	Drainage Area of Structures (Sq.Mi.)	Width (Miles)	Length to Centroid (Miles)	Geologic Constant (M)	Channel Characteristics: Miles above Durwood	Travel Time to Durwood (Hrs.)	Channel Loss to Durwood (%)	Unit Hydrograph Characteristics, Total Drainage Area: Time to Peak (Hrs.)	Peak Discharge Rate (cfs)	Recession Constant (d)	Drainage Area below Structures: Time to Peak (Hrs.)	Peak Discharge Rate (cfs)	Recession Constant (d)	Hydrologic Characteristics: No. of Rainfall Events 1941-50	Rainfall Total 1941-50 (In.)	Runoff Total 1941-50 (In.)
1	Durwood Laterals	74	10	5.8	14.1	.72	0	0	0.0	4	12,100	3.72	4	11,200	3.47	703	364.62	87.415
2	Caddo Creek	352	146	9.0	21.4	.74	1	0	0.4	10	18,000	6.46	10	9,900	5.52	973	368.80	77.024
3	Rock Creek	139	46	7.8	16.1	.81	21	7	2.8	7	14,500	4.83	7	9,500	4.12	906	405.42	100.612
4	Doughtery Laterals	84	10	6.4	6.8	.79	22	8	3.2	5	5,300	3.94	5	5,000	3.71	881	388.99	94.523
5	Chigley Sandy Creek	46	20	3.1	8.9	.77	36	13	5.2	1	8,800	2.88	1	5,100	1.90	945	409.36	100.409
6	Davis Laterals	50	10	4.1	11.2	.72	37	13	5.2	2	9,500	3.03	2	7,900	2.64	920	395.79	92.859
7	Wildhorse Creek	633	303	13.9	42.3	.72	39	14	5.6	24	37,200	7.50	23	17,500	6.35	942	353.57	72.032
8	Wynnewood Laterals	60	23	4.2	13.2	.72	46	16	6.4	2	12,000	3.35	2	7,100	2.50	763	379.86	89.815
9	Rush Creek	262	108	5.8	32.4	.64	65	23	9.2	4	29,700	5.95	4	16,400	5.01	1,001	382.25	70.932
10	Pauls Valley Laterals	47	11	3.5	12.2	.70	65	23	9.2	1	16,800	2.92	1	13,100	2.44	690	401.99	97.561
11	Cherokee Sandy Creek	65	38	5.4	8.8	.77	67	23	9.2	3	8,700	3.49	3	3,000	1.94	832	400.82	97.726
12	Peavine Creek	60	40	5.3	7.8	.72	75	26	10.4	3	7,300	3.35	3	2,200	1.50	800	389.22	92.361
13	Paoli Laterals	57	18	4.6	9.2	.45	83	29	11.6	2	10,200	3.26	2	7,000	2.61	739	377.18	64.056
14	Maysville Laterals	54	25	5.4	6.2	.56	104	37	14.6	3	6,000	3.17	3	3,000	2.08	704	365.38	73.862
15	Pinn Creek	65	37	4.8	8.0	.74	113	41	16.1	2	9,800	3.49	2	3,700	2.01	739	389.09	101.723
16	Wallville Laterals	49	14	4.8	7.3	.42	114	41	16.1	2	8,900	2.99	2	6,400	2.41	671	378.48	64.366
17	Criner Creek	67	36	3.9	10.0	.56	120	44	17.3	2	7,400	3.55	2	3,900	2.19	642	371.84	79.702
18	Lindsay Laterals	69	27	6.4	6.9	.38	130	48	18.8	5	5,400	3.60	5	3,100	2.70	546	382.83	61.775
19	Bradley Laterals	71	30	4.5	7.0	.38	140	52	20.3	2	7,200	3.65	2	4,700	2.66	546	382.83	61.629
20	Roaring Creek	69	41	5.2	10.9	.40	154	57	22.1	3	10,000	3.60	3	3,300	1.99	838	332.64	43.574
21	Alex Laterals	49	12	4.5	8.0	.36	154	57	22.1	2	8,300	2.99	2	6,300	2.48	721	338.67	42.262
22	Laflin Creek	30	19	3.1	5.8	.45	158	59	22.8	1	5,500	2.13	1	2,400	1.50	693	374.59	64.171
23	Winter Creek	44	24	4.6	7.9	.45	167	62	24.0	2	8,700	2.81	2	3,300	1.50	835	349.18	50.899
24	Chickasha Laterals	63	0	3.7	15.6	.32	179	67	25.8	1	25,300	3.44	1	25,300	3.44	742	333.22	33.363
25	Little Washita Creek	246	117	10.2	20.9	.27	180	68	26.1	13	17,300	5.83	12	8,200	4.70	948	336.20	27.564
26	Bitter Creek	104	52	5.6	9.3	.42	182	69	26.2	3	10,200	4.32	3	4,600	3.11	922	338.34	42.584
27	Line Creek	54	0	4.7	8.9	.21	194	75	27.3	2	10,400	3.17	2	10,400	3.17	798	338.32	23.470
28	Salt Creek	81	0	6.5	8.6	.53	199	78	27.8	5	7,100	3.88	5	7,100	3.88	843	343.65	53.806
29	Ionine Creek	67	31	5.8	7.9	.42	211	84	28.9	4	6,500	3.55	4	3,100	2.43	939	339.18	40.514
30	Verden Laterals	30	1	3.4	9.1	.11	212	84	28.9	1	11,400	2.13	1	11,400	2.06	929	328.08	10.693
31	Spring Creek	80	72	4.7	10.8	.17	222	90	30.1	2	12,900	3.86	1	1,900	1.50	1,015	319.92	15.591
32	Delaware Creek	39	36	4.1	7.0	.21	230	94	30.8	2	5,700	2.59	1	800	1.50	719	327.93	21.454
33	Sugar Creek	240	132	7.7	19.5	.06	234	96	31.2	7	17,300	5.79	6	6,700	4.38	896	293.56	5.263
34	Anadarko Laterals	113	0	7.3	21.7	.19	235	96	31.2	6	20,600	4.47	6	20,600	4.47	779	318.42	18.652
35	Tonkawa	46	19	4.2	10.7	.11	235	96	31.2	2	9,600	2.88	2	5,400	1.97	717	327.90	12.581
36	Pond Creek	324	7	12.5	19.4	.21	276	118	35.3	19	16,500	6.32	19	16,500	6.28	1,042	296.13	14.777
37	Fort Cobb Laterals	90	0	6.7	9.8	.21	276	118	35.3	5	8,900	4.06	5	8,900	4.06	762	304.41	17.886
38	Carnegie Laterals	67	0	8.6	8.4	.13	291	125	36.5	9	6,700	3.55	9	6,700	3.55	655	303.50	11.965
39	Saddle Mountain Creek	106	40	5.8	18.7	.44	300	130	37.3	4	16,400	4.35	4	9,800	3.52	835	302.13	31.174
40	Rainy Mountain Creek	311	220	15.8	13.1	.39	311	136	38.2	32	10,000	6.25	27	2,200	4.09	976	279.03	19.682
41	Mountain View Laterals	84	25	3.6	12.2	.19	311	136	38.2	1	18,100	3.94	1	12,800	3.33	797	284.28	13.167
42	Conden Laterals	85	10	7.9	9.6	.25	324	143	39.2	7	8,600	3.96	7	8,200	3.75	596	281.39	18.388
43	Cavalry Creek	104	56	8.8	10.6	.34	345	153	40.8	9	9,100	4.32	8	3,600	2.95	729	267.87	20.190
44	Weatherford Laterals	87	0	6.1	14.8	.31	345	153	40.8	4	14,500	4.00	4	14,500	4.00	892	286.21	20.805
45	Boggy Creek	119	71	6.4	18.1	.33	350	156	41.3	5	15,300	4.51	4	4,900	2.85	764	264.66	18.842
46	Clinton Laterals	112	59	6.7	11.9	.31	363	163	42.4	5	10,900	4.45	5	4,500	3.13	797	287.36	21.521
47	Beaver Creek	77	53	3.9	12.3	.19	383	174	44.0	2	9,200	3.79	2	3,300	1.72	685	254.65	10.733
48	Butler Laterals	90	14	3.8	13.5	.14	384	174	44.0	2	9,500	4.06	2	8,500	3.77	721	278.47	9.552
49	Barnitz Creek	265	147	10.4	17.0	.06	389	175	44.2	13	14,500	5.97	12	5,500	4.53	765	244.12	2.892
50	Turkey Creek	75	48	5.4	10.7	.18	397	178	44.8	3	10,800	3.74	3	3,200	1.98	835	264.73	10.055
51	Oak Creek	52	36	4.1	15.3	.18	400	178	44.8	2	13,200	3.10	2	3,200	1.50	747	247.30	10.157
52	Carpenter Laterals	85	0	6.3	12.5	.12	401	179	45.1	4	13,100	3.96	4	13,100	3.96	631	244.59	6.403
53	Panther Creek	29	14	2.7	8.5	.23	429	187	46.7	1	5,900	2.07	1	3,300	1.50	659	251.14	12.594
54	Quartermaster Creek	176	110	10.5	17.2	.08	436	189	47.1	13	21,400	5.25	12	4,600	3.52	644	238.61	4.420
55	Hammon Laterals	35	3	3.4	7.9	.10	436	189	47.1	1	9,800	2.40	1	9,800	2.27	535	250.90	5.469
56	White Shield Creek	26	12	2.4	10.3	.24	443	190	47.3	1	5,400	1.88	1	3,000	1.50	726	249.05	12.099
57	Nine Mile Creek	39	19	3.8	9.8	.18	450	192	47.8	2	6,700	2.59	2	3,000	1.50	747	249.88	9.452
58	Kiowa Creek	29	16	3.1	9.3	.23	452	193	47.9	1	9,200	2.07	1	3,400	1.50	723	252.65	12.265
59	Berring Laterals	53	19	4.2	10.1	.15	452	193	47.9	2	9,000	3.13	2	5,600	2.37	703	268.68	10.170
60	Sandstone Creek	101	73	5.6	13.3	.20	458	195	48.4	3	15,000	4.27	3	3,000	1.99	806	269.17	13.229
61	Strong City Laterals	72	39	5.5	8.6	.14	458	195	48.4	3	8,900	3.67	3	3,600	2.31	755	270.67	9.896
62	Dead Indian Creek	56	36	4.3	9.2	.12	476	200	49.4	2	8,600	3.23	2	2,900	1.50	721	260.07	8.343
63	Cheyenne Laterals	46	18	3.5	5.3	.26	481	201	49.6	1	6,800	2.88	1	5,200	1.98	717	275.44	20.435
64	Sargent Creek	26	15	2.9	6.5	.28	481	201	50.0	1	5,300	1.88	1	2,400	1.50	590	278.74	23.754
65	Croton Creek	52	49	4.7	5.8	.17	489	203	50.4	2	6,600	3.10	1	800	1.50	803	264.35	11.705
66	Rush Creek	76	47	4.9	10.5	.06	494	204	50.8	3	8,300	3.77	3	2,900	2.09	594	260.07	5.876
67	Durham Laterals	106	70	7.3	17.5	.13	494	204	50.8	6	16,300	4.35	5	4,300	2.43	632	244.79	7.261
68	Allison Laterals	133	89	8.6	10.4	.06	512	208	52.5	9	8,400	4.75	8	2,100	2.79	718	258.77	4.144
69	Gageby Laterals	121	71	4.9	18.9	.06	528	212	53.7	3	15,600	4.59	3	5,400	3.04	671	240.19	3.089
70	Washita Laterals	234	155	8.5	18.8	.06	528	212	53.7	8	18,000	5.75	7	4,700	3.83	658	240.50	3.144

Outflow from structures was calculated by routing precipitation, runoff, and evaporation through a reservoir. In order to facilitate economy of operations, the physical characteristics of an average structure of one-square-mile drainage area were developed. These characteristics were based on 242 structures in the Washita basin with a combined drainage area of 800 square miles.

Rainfall, runoff, and evaporation for the 10-year period on each of the 70 subwatersheds were routed through the one-square-mile structure and the discharge multiplied by the number of square miles behind structures. The structure outflow was then routed down the river to the main stem stations.

Flood hydrographs reflecting the FRS program were developed by multiplying the routed unit hydrographs from the uncontrolled area by precipitation excess and adding the routed outflow from structures.

Figure 20 is a representative sample of the flood hydrographs with and without the FRS program.

Discussion

Flow-duration, flow-volume, and flow-frequency curves were developed for each river station for the observed 10-year period, the predicted 10-year period without structures, and the predicted 10-year period with structures. Flow-duration and flow-volume curves at Carnegie are presented in Figure 21. The difference in the observed and predicted without structures flow-duration curves at low discharge rates is due to errors in separating base flow from surface flow.

Since the period of record was short, a partial duration series was used to determine frequency curves from mean daily discharge rates. The curves for Carnegie are shown on Figure 22. Because rate of change of the flow-duration and flow-volume curves are more indicative of changes that have taken place, curves of this type have been drawn for each river station. The curves for the Carnegie river station are shown on Figures 23 and 24.

Analysis of the frequency curves and distribution curves of all stations used in the study have been completed. Following are the conclusions reached:

Flow-frequency - The period of record used in the study was too short to define the changes in flood frequency caused by an FRS program for return periods greater than 10 to 15 years.

Large storms are overpredicted and small storms underpredicted. In the upper reaches of the watershed where rainfall is lighter and the geologic constant lower, the predicted frequency curve for the 10-year period without structures plots lower than the observed record. In the lower reaches, the predicted frequency curves for the 10-year period with structures plots above the observed record. In the central part

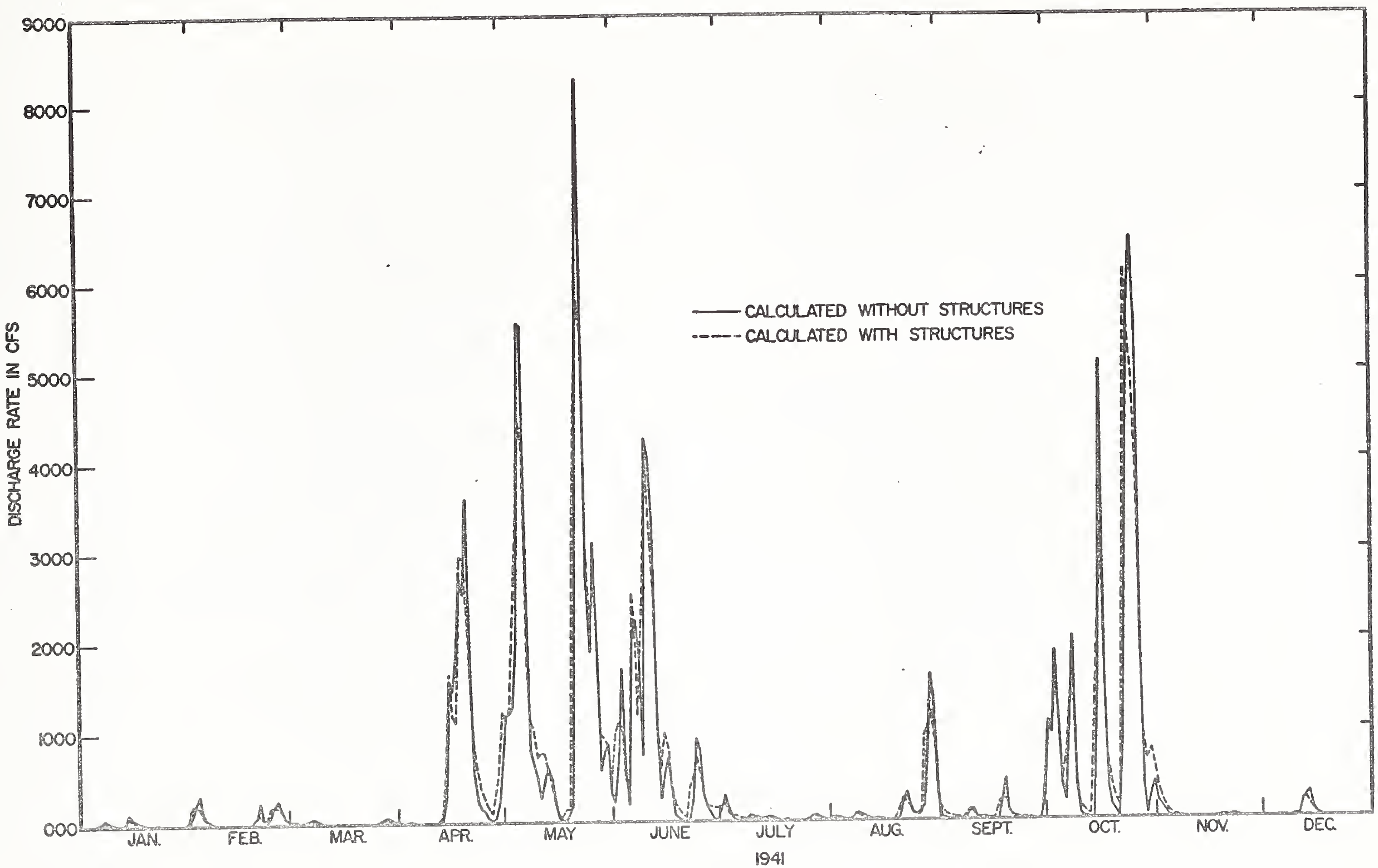

Figure 20
Mean daily flow rates for Washita River at Carnegie, Oklahoma

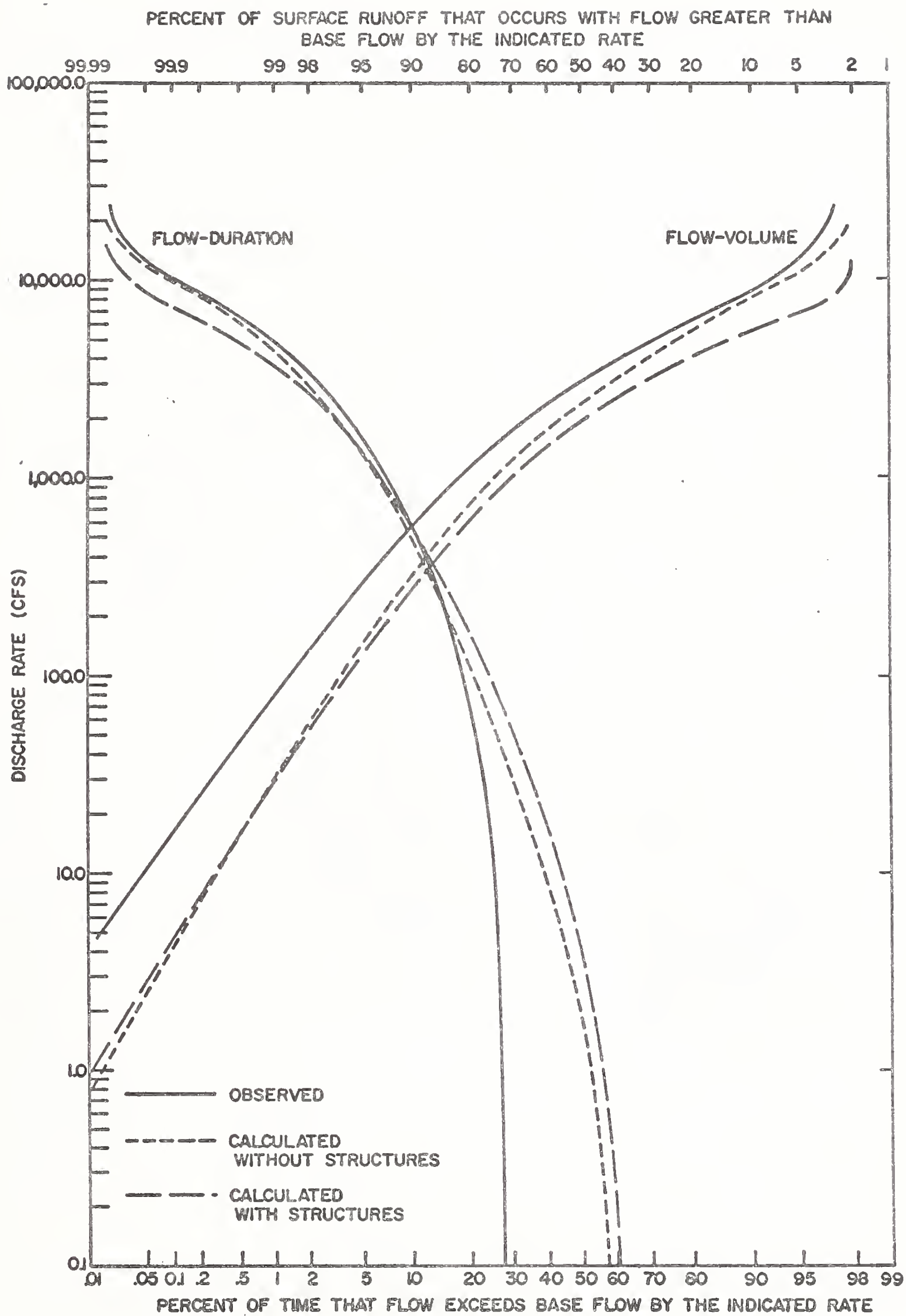

Figure 21

Flow-duration and flow-volume curves for Washita River at Carnegie, Oklahoma 1941-1950

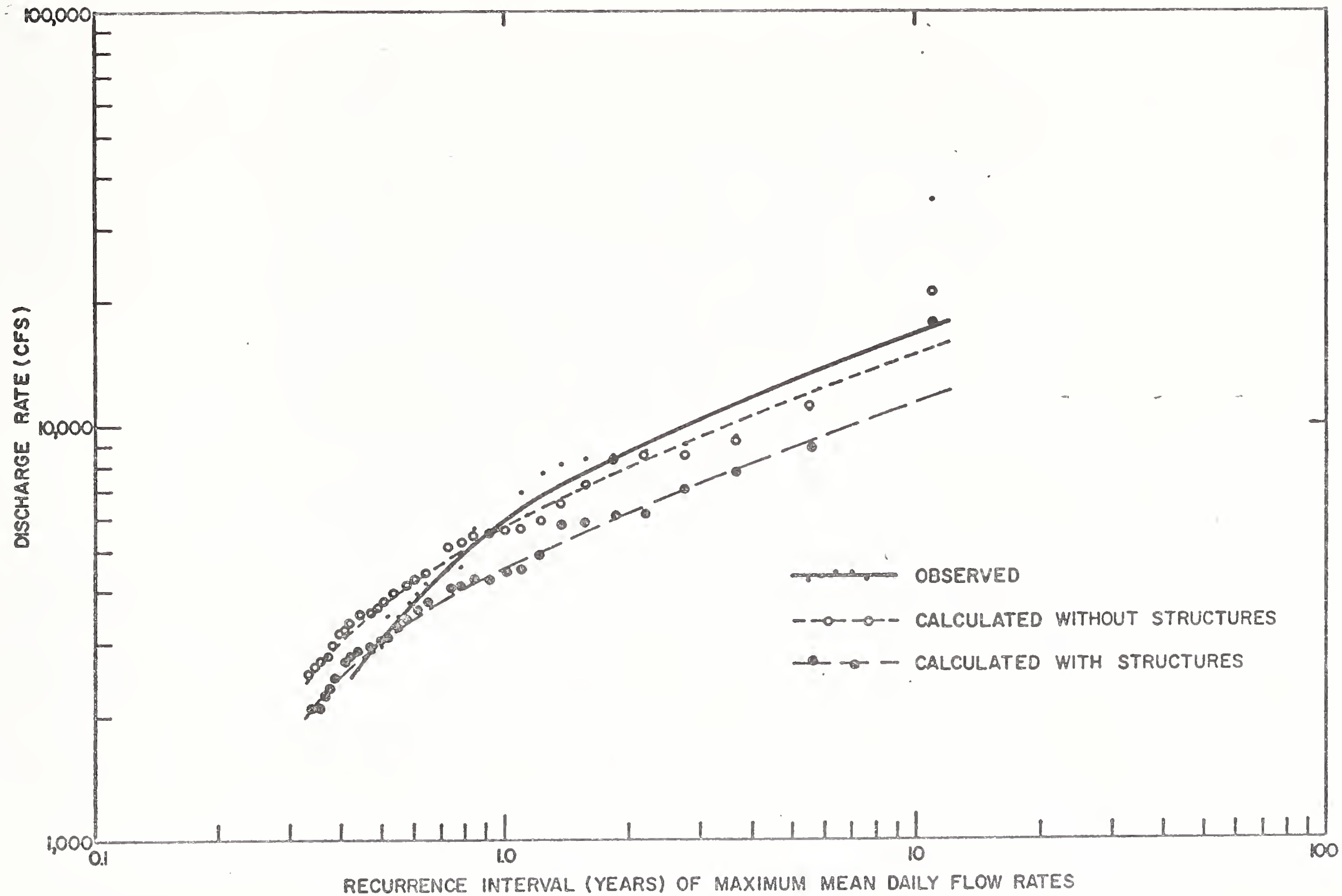

Figure 22
Flow-frequency curves for Washita River at Carnegie, Oklahoma
1941-1950

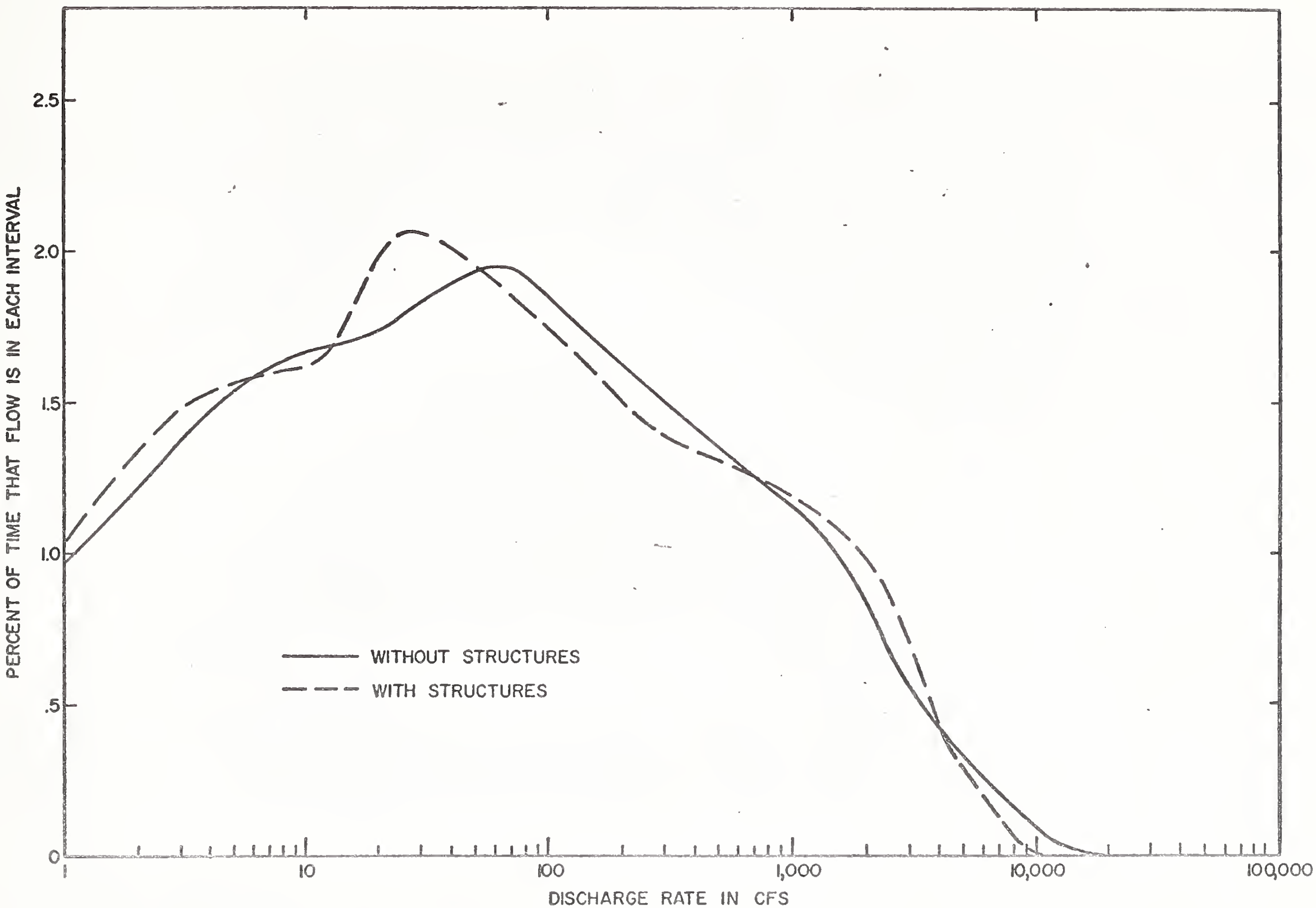

Figure 23
Distribution of flow-duration by rate at Carnegie, Oklahoma
1941-1950

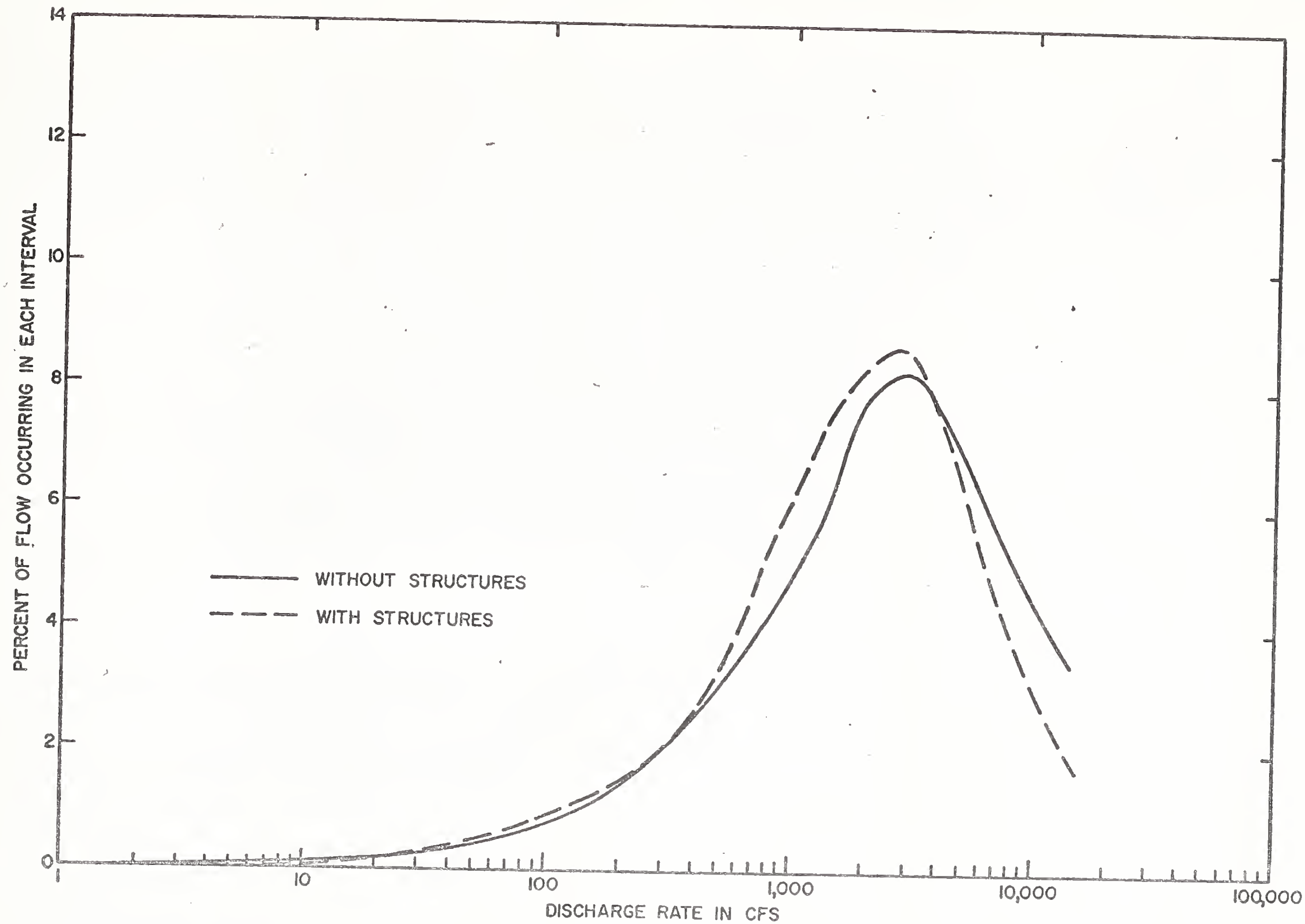

Figure 24
Distribution of flow-volume by rate at Carnegie, Oklahoma
1941-1950

of the watershed, the correspondence is good. The trend is associated with differences in rainfall across the watershed and with differences in the geologic constants. These deviations will not affect the relative difference between the frequency curves of the without and with structures conditions.

The frequency curve obtained from the predicted mean daily peaks without structures is used as a base to calculate the percent reduction in peak rates for each station caused by the structures program. These reductions for various frequencies of occurrence are shown in Table 16. Blank spaces in the table are due to a poor plotting pattern of the points. The frequency of occurrence was used as the weight factor to obtain the weighted average reduction for each station. The percent of the drainage basin that was assumed to lie behind flood retarding structures, weighted by the volume of surface runoff at the station, is also shown in Table 16. Figure 25 is a plot of the percent of the watershed behind floodwater retarding structures vs. the reduction in peak.

The 1941-50 data presented in Table 16 show a relatively constant reduction in peaks. However, there are two deviations from this trend. In areas of low runoff, Cheyenne and Clinton, there is a gradual decrease in the percent reduction in peaks with a decrease in the frequency of occurrence. This is caused by the combination of a low volume of runoff and proportionately high evaporation loss. The other exception is in the lower reaches at Pauls Valley and Durwood where the large events show almost no reduction. This was caused by emergency spillway flow which did not occur at all in the upstream reaches and only very seldom in the central portion. This would indicate a constant reduction in peaks with respect to the frequency of occurrence unless the event is large enough to produce emergency spillway flow.

Flow-distribution - The distribution of flow duration by rate for the Carnegie station is portrayed on Figure 23. For this mid-reach station the flow-duration for the without structures condition is approximately equally distributed about a flow rate of 70 cfs. Curves for upstream stations (dryer areas) show a greater percentage flow-duration at smaller rates, and curves for downstream stations (wetter areas) show the greater percentage at the larger rates. Table 17 shows the mean annual surface flow rate, the discharge rate at which peak duration occurs, and the ratio of the rate at peak occurrence to the mean annual flow.

The distribution of flow-duration by rate curves shows that the FRS program causes an increase in the duration of flow in the central range of flow rates. There is a corresponding decrease in the duration of flow in the extreme upper range. At the lower end, the durations are approximately the same. The point of intersection, or cross-over point, at about 3,550 cfs, of the two curves on Figure 23 is a measure of the effect of structure installation on the flow-duration characteristics of the Washita River. Table 17 lists the cross-over point and its ratio to the mean annual flow for all stations. The point at which

Table 16.--Percent reduction in mean daily flow peaks

Station	Frequency of occurrence 2	1	.5	.2	.1	Weighted average	Percent of drainage area behind structures*
Cheyenne	58.7	50.6	48.3	43.9	40.4	53.9	62.5
Clinton	35.1	38.9	47.5	42.1	33.7	38.1	54.2
Carnegie	26.3	24.6	24.0	24.3	24.8	25.4	49.3
Tabler	18.9	18.9	17.6	20.0	21.3	18.3	38.1
Pauls Valley	16.4	24.6	28.6	--	--	20.8	41.4
Durwood	25.2	26.6	24.6	12.7	0.0	24.2	40.6

*Weighted by volume of runoff

Table 17.--Distribution of flow-duration by rate

Station	Mean annual surface flow rate (cfs)	Peak duration: Discharge rate (cfs)	Peak duration: Ratio to mean annual	Cross-over point: Discharge rate (cfs)	Cross-over point: Ratio to mean annual
Cheyenne	31	10	.32	1,350	43.5
Clinton	91	30	.33	2,200	24.2
Carnegie	230	70	.30	3,550	15.4
Tabler	435	70	.16	4,500	10.3
Pauls Valley	645	300	.47	6,300	9.8
Durwood	1,625	700	.43	16,300	10.0

the two curves separate at the lower rates of flow is shown more clearly on Figure 24. However, the point at which they separate is poorly defined. The ratio of the cross-over point to the mean annual flow is plotted vs. the average area (sq.mi.) inches of runoff in Figure 26.

The FRS program causes an increase in the volume of flow that occurs at flow rates in the central range, and a decrease at the upper extreme of flow rate. The distribution of flow-volume by flow rate for the Carnegie station is typical of the Washita River and is selected for portrayal on Figure 24. Table 18 lists the discharge rate at which the peak volume of flow occurs for the without-structures condition and the with-structures condition, and the ratio of these two rates. The average of these ratios shows that the FRS program will reduce the rate at which the peak volume occurs by about one-third.

The percent of flow that will occur during this peak interval is shown in Table 18 for the two watershed conditions. The ratio of these two peaks, also shown in Table 18, averages about 1.10. In other words, the volume of runoff during the peak flow with the FRS program in effect is about 10 percent higher than without the program and occurs at a lower rate of flow.

The distribution of flow-duration and volume by rate curves shows that intermediate range flows were more frequent and of longer duration after the FRS program was in effect. Changes resulting from these structures at the various river stations can be obtained from Tables 16-19 and Figures 25 and 26.

Volume of surface runoff - The model used in the study did not take into account any seepage losses from the structures for two reasons:

1. The volume-rate of seepage from such structures is not known.
2. The destination of such seepage is not known.

All other hydrologic phenomena were taken into account.

Shown in Table 19 are the average watershed inches of runoff per year for the 10-year period without structures. Also shown in Table 19 is the ratio of the volume of runoff from the watersheds with the FRS program in effect to the volume without a program. The upstream reaches show about a 25 percent reduction in total runoff following the construction of flood control reservoirs, whereas the downstream reaches show less than 5 percent. Explanation for the high losses in the upstream reaches lies in the coupling of high evaporative losses and low surface runoff with a relatively large sediment pool. The lower reaches of the basin have a lower evaporative loss, higher surface runoff, and relatively small sediment pool.

Table 18.--Distribution of flow-volume by rate

Station	Discharge rate at peak interval			Percent of flow occurring during peak interval		
	Untreated condition (cfs)	SCS condition (cfs)	Ratio	Untreated condition	SCS condition	Ratio
Cheyenne	--	--	--	--	--	--
Clinton	2,500	1,600	.64	7.5	8.4	1.12
Carnegie	3,000	2,500	.83	8.3	8.9	1.07
Tabler	4,700	3,300	.70	7.8	8.4	1.08
Pauls Valley	6,700	4,100	.61	7.2	7.7	1.07
Durwood	19,000	12,000	.63	7.1	8.1	1.14
			.68			1.10

Table 19.--Station runoff

Station	Drainage area (sq.mi.)	Average runoff per year (inches)	Average area-inches of runoff (sq.mi.in.)	Ratio of runoff treated to untreated conditions
Cheyenne	794	.52	413	.725
Clinton	1,977	.62	1,226	.788
Carnegie	3,129	1.00	3,129	.880
Tabler	4,700	1.26	5,930	.927
Pauls Valley	5,330	1.65	8,794	.951
Durwood	7,202	2.98	21,462	.971

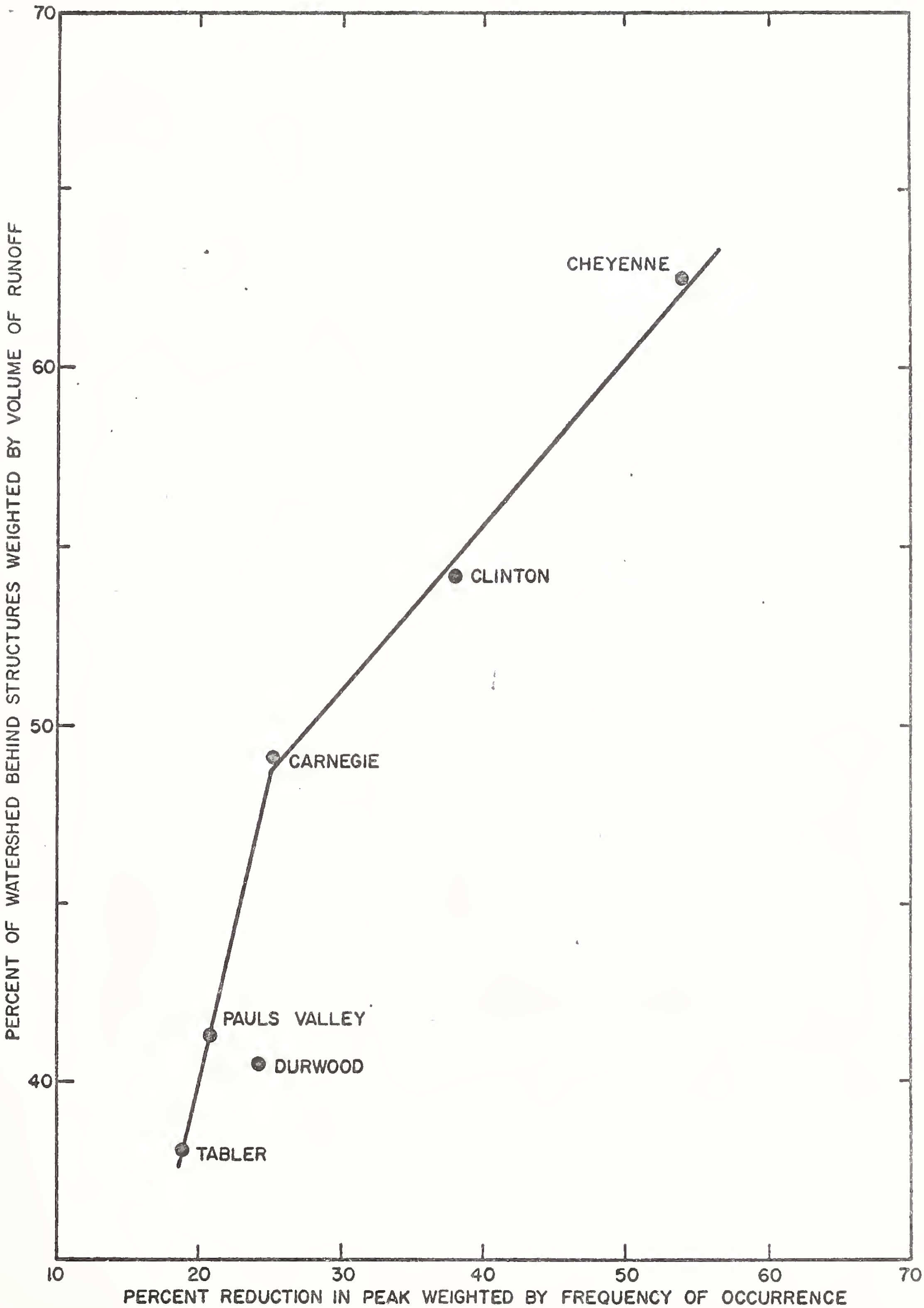

Figure 25
Percent of watershed protected vs. reduction in peak

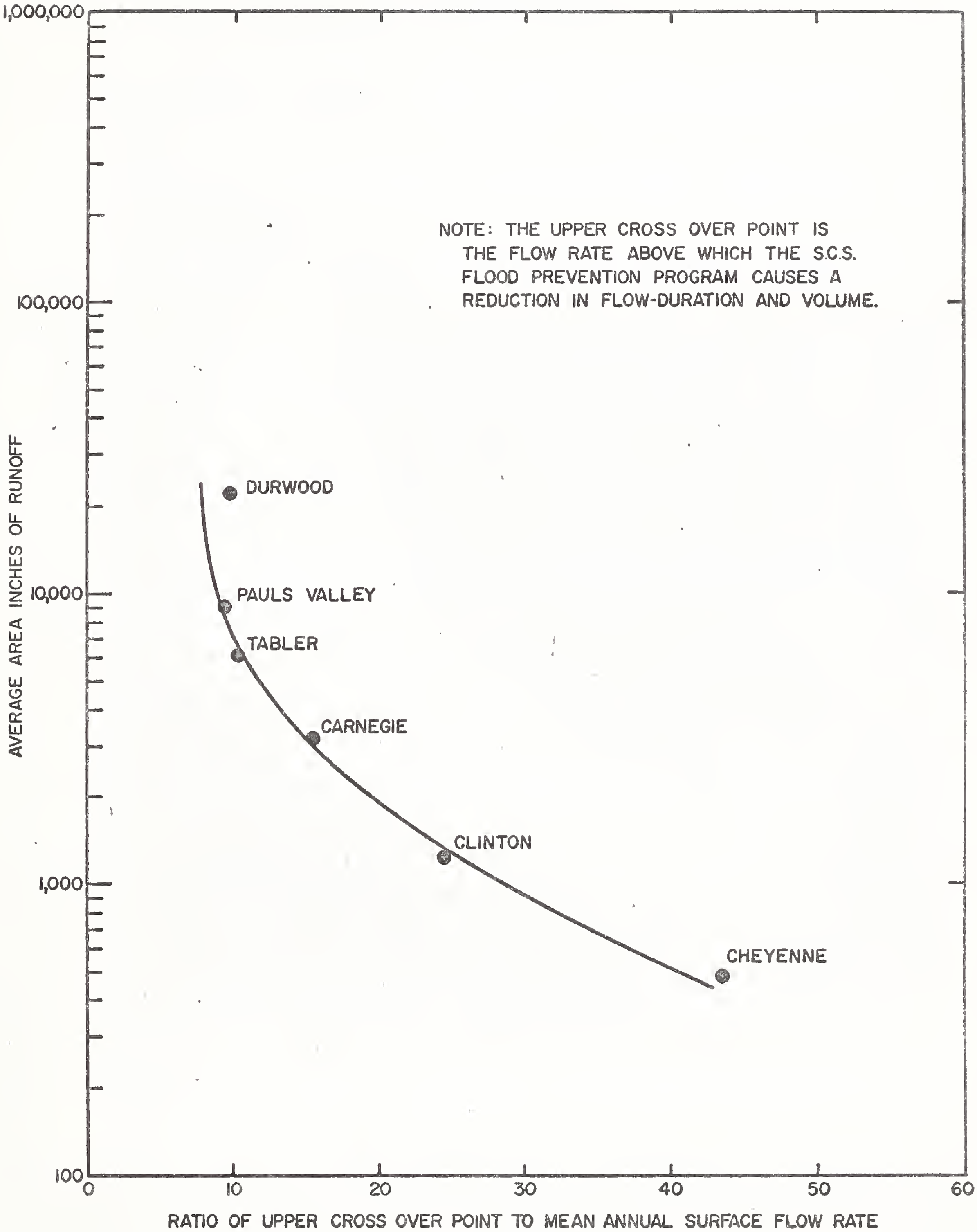

Figure 26
Average area inches of runoff vs. the ratio of the upper cross over point to the mean annual surface flow rate

LIST OF PUBLICATIONS

Southern Plains Watershed Research Center
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1. Rhoades, Edd D. A mobile double-tower sprinkling infiltrometer. Journal of Soil and Water Conservation. 16(4): 181-182. 1961.

2. Rhoades, Edd D. and Gamble, M. D. Researchers study flooding tolerance of grasses. Oklahoma Farmer Stockman. 75(2): 24. 1962.

3. Hartman, Monroe A. and Ree, William O. Study downstream effects of upstream watershed improvements. Soil Conservation Magazine. April 1963.

4. Hartman, Monroe A., Yost, Coyd Jr., Rhoades, Edd D., DeCoursey, Donn G., Allen, Paul B., Schoof, Russell R., and Nicks, Arlin D. Washita River watershed research progress report. Mimeo. 1963.

5. Rhoades, Edd D. Grasses and flood survival. Agricultural Engineering. 44(1): 22-23. 1963.

6. Rhoades, Edd D., Locke, Lowell F., Taylor, Howard M., and McIlvain, E. H. Water intake on a sandy range as affected by twenty years of differential cattle stocking rates. Journal of Range Management. 17(4): 185-190. 1964.

7. Rhoades, Edd D. Inundation tolerance of grasses in flooded areas. Transactions of ASAE. 7(2): 164-166, 169. 1964.

8. Gamble, M. D., and Rhoades, Edd D. The effect of shorline fluctuations upon grasses associated with upstream flood prevention and watershed protection. Agronomy Journal. 56(1): 21-23. 1964.

9. DeCoursey, Donn G. Water yield computations. Transactions of ASAE. 8(3): 367-370. 1965.

10. DeCoursey, Donn G. A runoff hydrograph equation. ARS 41-116. 1965.

11. Nicks, Arlin D. Field evaluation of rain gage network design principles. IASH Proceedings. Paper 67: 82-93. 1965.

12. Rhoades, Edd D., and Cox, Maurice B. Water intake and runoff rates of two small watersheds in relation to cropping treatment. ASAE Transactions. 9(3): 340-342. 1965.

13. Nicks, Arlin D. and Hartman, Monroe A. Variation of rainfall over a large gaged area. ASAE Transactions. 9(3): 437-439. 1966.

14. Edens, Carlton D., Padgham, Wilber W., and Schoof, Russell R. A cantilevered wire-weight water level gage. ARS 41-128. 1966.

15. Welch, Norman H., Shockey, W. R., Hartman, Monroe A., and Stone, J. F. Soil moisture record, Chickasha, Oklahoma. OSU Process Series P-536. May 1966.

16. Hunt, Charles G., Price, Harold E., Schoof, Russell R., and Hartman, Monroe A. Giant sand point for a bubble gage orifice housing. ARS 41-129. 1966.

17. Watershed Research in the Southern Great Plains. Brochure about work at the Southern Great Plains Research Center, SWC-ARS-USDA. 1966.

18. Allen, Paul B. A comparison of topographic mapping methods. Transactions of ASAE. 9(6): 782-783. 1966.

19. Hartman, Monroe A., Ree, William O., and Nicks, Arlin D. Effect of conservation programs on the main stem of a river basin. Proceedings of the 22nd Annual Meeting of SCSA. 1966.

20. Schoof, Russell R. and DeCoursey, Donn G. Conveyance of irrigation water in a natural channel. Proceedings of the 2nd Annual American Water Resources Conference. 1966.

21. Allen, Paul B. and Welch, Norman H. Sediment transport of streams in the Washita River Basin in Caddo and Grady Counties, Oklahoma. Transactions of AGU. 3(3): 777-784. 1967.

22. Hartman, Monroe A., Ree, William O., Schoof, Russell R., and Blanchard, Bruce J. Hydrologic influences of a flood control program. Proceedings of the ASCE. 93(HY3). May 1967.

23. Nicks, Arlin D. A computer mapping method for analysis and summary of rainfall data. ARS 41-135. 1967.

24. Rhoades, Edd D. Grass survival in flood pool areas. Journal of Soil and Water Conservation. 22(1). 1967.

25. Allen, Paul B., and Welch, Norman H. Variations of sediment transport in the Washita River. IUGG Transactions. 1968.

26. Blanchard, Bruce J. Anchoring automobile bodies for streambank protection. ARS 41-138. 1968.

27. Schoof, Russell R., Hartman, Monroe A., and Hunt, Charles G. Determining streamflow abstractions from antecedent conditions. IUGG Transactions. 1968.

28. Hartman, Monroe A. Upstream flood control effects on river basin hydrology. Agricultural Engineering. 49(2): 80-81. 1968.